NO. WW4
DECEMBER 1966

JOURNAL of the

Waterways and Harbors Division

PROCEEDINGS OF THE
AMERICAN SOCIETY
OF CIVIL ENGINEERS

VOLUME 92

BASIC REQUIREMENTS FOR MANUSCRIPTS

This Journal represents an effort by the Society to deliver information to the reader with the greatest possible speed. To this end the material herein has none of the usual editing required in more formal publications.

Original papers and discussions of current papers should be submitted to the Manager of Technical Publications, ASCE. The final date on which a discussion should reach the Society is given as a footnote with each paper. Those who are planning to submit material will expedite the review and publication procedures by complying with the following basic requirements:

1. Titles should have a length not exceeding 50 characters and spaces.

2. A 50-word summary should accompany the paper.

3. The manuscript (a ribbon copy and two copies) should be double-spaced on one side of 8½-in. by 11-in. paper. Papers that were originally prepared for oral presentation must be rewritten into the third person before being submitted.

4. The author's full name, Society membership grade, and footnote reference stating present employment should appear on the first page of the paper.

5. Mathematics are reproduced directly from the copy that is submitted. Because of this, it is necessary that capital letters be drawn, in black ink, 3/16-in. high (with all other symbols and characters in the proportions dictated by standard drafting practice) and that no line of mathematics be longer than 6½-in. Ribbon copies of typed equations may be used but they will be proportionately smaller in the printed version.

6. Tables should be typed (ribbon copies) on one side of 8½-in. by 11-in. paper within a 6½-in. by 10½-in. invisible frame. Small tables should be grouped within this frame. Specific reference and explanation should be made in the text for each table.

7. Illustrations should be drawn in black ink on one side of 8½-in. by 11-in. paper within an invisible frame that measures 6½-in. by 10½-in.; the caption should also be included within the frame. Because illustrations will be reduced to 69% of the original size, the capital letters should be 3/16-in. high. Photographs should be submitted as glossy prints in a size that is less than 6½-in. by 10½-in. Explanations and descriptions should be made within the text for each illustration.

8. Papers should average about 12,000 words in length and should be no longer than 18,000 words. As an approximation, each full page of typed text, table, or illustration is the equivalent of 300 words.

Further information concerning the preparation of technical papers is contained in the "Technical Publications Handbook" which can be obtained from the Society.

Reprints from this Journal may be made on condition that the full title of the paper, name of author, page reference (or paper number), and date of publication by the Society are given. The Society is not responsible for any statement made or opinion expressed in its publications.

This Journal is published by the American Society of Civil Engineers. Publication office is at 2500 South State Street, Ann Arbor, Michigan. Editorial and General Offices are at 33 West 39 Street, New York 18, New York. $4.00 of a member's dues are applied as a subscription to this Journal.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

WATERWAYS AND HARBORS DIVISION
COMMITTEE ON PUBLICATIONS
Ellsworth I. Davis, Chairman; John M. Buckley; Joseph M. Caldwell
Jay V. Hall, Jr.; Evan W. Vaughan

CONTENTS

December, 1956

Papers

Number

Anchorages for Large Tainter Gates
by A. H. Kenigsberg 1119

The Apalachicola River Basin Project
by C. P. Lindner 1120

The Development of the Cumberland River Basin
by Frank P. Gaines and John T. Dennison 1121

Modern Towboat and Barge Design
by Harry B. Dyer 1122

Current Trends in Ohio River Traffic and Equipment
by Charles F. Michiels, William F. Lail, and Robert E. Mytinger .. 1123

Discussion 1124

Local Flood-Protection Projects, Ohio River Basin
by S. M. Bailey, R. E. Karlen, and Harry Pockras 1128

DIVISION ACTIVITIES

News 1956-27

Copyright 1956 by the American Society of Civil Engineers.

Paper 1119 WW 5

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

ANCHORAGES FOR LARGE TAINTER GATES

A. H. Kenigsberg,* M. ASCE
(Proc. Paper 1119)

ABSTRACT

Novel type anchorages whose distinguishing features and economy derive from a single tie girder placed centrally within the spillway pier, a single anchor at the upstream end of the girder, and its downstream end in a concentric grouping with the inclined side arms of adjacent tainter gates.

INTRODUCTION

A novel type anchorage for tainter gates has been developed in the Nashville District, Corps of Engineers. It is considered to be an improvement over previous designs and is being presented at this time to acquaint the engineering profession with the basic features of the new arrangement.

The versatility of tainter type gates, in general, has by now been thoroughly established. Not only are the operating characteristics relatively elementary in that the basic requirement is provision merely for hoisting a portion of its dead weight, but also the structural design of the moving portions of the gate is a relatively simple procedure in mechanics. The bearing problems which are so troublesome in vertical lift gates simply do not exist at the upstream end of tainter gates. Effective sealing is also easily accomplished. However, at the downstream ends, where the considerable hydrostatic thrust of modern large gates is concentrated at a single point and must be transferred from the trunnions to the masonry piers, there arise rather complex problems in accomplishing the load transfer. While tainter gates proper have been treated in engineering literature, their anchorages have so far received but scant attention. In the brief time available here, it is our object to fill the gap somewhat by looking into the engineering problems involved, to cover the

Note: Discussion open until May 1, 1957. Paper 1119 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 5, December, 1956.

* Civ. and Structural Engr., Nashville Dist., Corps of Engrs., U. S. Dept. of the Army, Nashville, Tenn.

basic methods of solving them, and to compare the characteristics of the new anchorage with those of the conventional type.

Basic Problems

The basic structural problem in tainter gate anchorage design is how to transfer the hydrostatic load on the gates from the trunnions to its optimum location in the masonry of the spillway in as efficient a manner as possible.

It will be helpful to give first an idea of the magnitude of the forces involved at the gate trunnions. In round figures, the direct hydrostatic thrust is frequently 1,000,000 lbs. and over at each trunnion for the size of tainter gates under consideration. When the gate arms are inclined, which, incidentally, is by far the most economical and preferred arrangement, approximately a third of this longitudinal force is converted into lateral thrust at the trunnions.

When the loads from adjacent gates are balanced, their lateral forces are also balanced within the steel work of the anchorage and the transfer of the balanced longitudinal loads to the masonry is a relatively routine procedure of design. However, at the end gates under all loading conditions and at intermediate piers of the spillway, when the water loads on adjacent gates are unbalanced, we have, acting on each trunnion pin, an eccentric primary thrust comprising a longitudinal component of about 1,000,000 lbs. and a lateral component of about 330,000 lbs.

Of course, as a result of built-in frame rigidity between the gate proper and its side arms, there are induced at the trunnions secondary lateral forces which sometimes are additive to the effects of the primary forces. However, their magnitude is not sufficient to affect design considerations here.

While the moments developed within the anchorage steel by the primary forces at the trunnions are absorbed within the anchorage steel and are of no direct concern in this paper, nevertheless they are worth underscoring, in passing, since their magnitude is quite impressive and has important bearing on anchorage design and pier arrangement at the downstream end. Since trunnions are basically cantilevered supports, each 1,000,000 lbs. of hydrostatic thrust becomes a 3,000,000 ft.-lb. moment on either side of the ordinary anchorage frame, assuming a minimum distance of 3 feet from trunnion bearing to the embedded tension members.

Since the lateral components of the forces on the trunnions are not the governing factor in anchorage design and are best provided for as near the point of their application as practicable for any type anchorage, it will simplify our approach if they are deferred for consideration until later. There remains then the longitudinal component of a million pounds on either side of the pier as the principal force that must be provided for in design and which is of prime concern in this presentation.

There are but two basic ways of transmitting the hydrostatic thrust on the tainter gate from its trunnions to the pier masonry. One, in which an intermediate or transverse trunnion girder is employed, has its longitudinal tension members, which are located near each face of the pier, bonded into the concrete lying directly upstream of this girder. The other method, which may or may not employ transverse girders, has its tension members coated for a considerable distance so as to prevent them from bonding to the surrounding concrete in order to deliver their stress to anchors embedded nearer the upstream end of the spillway piers.

The first method of fully bonded side members, is direct, economical, and simple to develop, but it results in applying the reactions due to gate thrust near the downstream face of the pier where its contribution to dam stability is relatively small and where objectionable tensile stresses may develop within the mass concrete of the pier. For these reasons, this method of anchoring is not now generally entertained when large gates are involved.

There are a number of ways of transmitting the water thrust on the gate by the second method in which the side tie members do not part with their stress until some predetermined distance is reached well upstream of the trunnions.

There are also a great variety of major details of design in tainter gate anchorages that have been developed and used by the Corps of Engineers and by other agencies, both public and private. It would be impossible to discuss the many variations in the time available for the presentation of this paper. The discussion will be, therefore, limited to two main types of anchorages, both of which have been used for large tainter gates designed in the Nashville District. For want of a better name, we will call the ordinary type the "conventional" anchorage, and the novel type the "single girder" anchorage.

Conventional Anchorages

The usual conventional arrangement, such as the anchorage frame illustrated in Figure 1, employs an exposed transverse trunnion girder and two embedded longitudinal built-up members, one near each face of the pier, as the means of transferring the hydrostatic pressures on the gate to upstream anchor blocks. The trunnion girder is necessarily cantilevered outside the faces of the masonry pier in order to accommodate the trunnion pins and their bearing assemblies. Due to the resulting leverage, the reaction developed in the side tension members when gate loads are unbalanced becomes about 50 percent greater than the actual water thrust on the trunnion pin. For the permissible unit stress of 18,000 psi and a trunnion reaction of 1,000,000 lbs., the area of steel in each side member, for direct tension alone, is thus increased from the 55 square inches required for carrying the thrust when adjacent gates are fully loaded to 83 square inches when the gate loads are unbalanced. Consequently, a proportionate increase is required also in the size of the anchor blocks at the upstream ends of the side members. Further, the exposed transverse girder, which supports the trunnion bearings and is preferably of box type design, is a very husky member subject to a shearing force of 1,000,000 lbs., a bending moment of over 3,000,000 ft.-lbs. due to cantilever action, and an axial compression of about 330,000 lbs. due to the inclination of the gate arms.

In another arrangement of conventional design, the side tension members of the frame, in lieu of single girders, are made up of groups of bars, wrapped around or otherwise joined to the transverse trunnion girder. These bars are often prestressed in order to prevent their elongation under load and consequent objectionable movement of the trunnion girders. In either scheme, of built-up girders or groups of bars, these longitudinal tie members are coated for a considerable portion of their length to prevent their bonding to the concrete and to insure delivery of their stress to the anchor blocks located well upstream in the spillway piers for optimum dam stability.

The usual arrangement of the trunnions proper for conventional anchorage

frames is to employ individual pins set in trunnion hubs and adjustable bearing yokes at each end of the transverse girder.

For a complete picture of the existing stress conditions in these anchorages, it is necessary to observe in passing that, in addition to stresses due to frame rigidity, some torsional stresses also are developed in the trunnion girders, first, as a result of the setting of the achorage frame within the masonry and, especially, because of the eccentricity of the vertical trunnion reactions with respect to the trunnion girder during gate operation. However, these stresses are not coincident with maximum loading conditions and, consequently, are properly ignored in design.

Single Girder Anchorages

The novel type anchorage developed in the Nashville District was adopted for the 45 ft. wide by 41 ft. high tainter gates of the recently completed Old Hickory Dam on the Cumberland River just above Nashville.

As shown in Figure 2, the new arrangement employs a single longitudinal tie girder with a single anchor block at its upstream end placed centrally within the spillway pier. In this fashion, the water thrust on the gate is transmitted directly to the center of the pier where it belongs for optimum resistance under any condition of loading. It will be noted from the drawing that the inclined arms of adjacent gates meet the central girder at a common point just outside its downstream end. As a result of the concentric intersection of the three members involved, their stresses are axial only under normal conditions of balanced loading and the maximum total stress in the central longitudinal girder is never larger than that occasioned by the actual hydrostatic load on two halves of the adjacent gates. It can be noted also that a relatively short pin, extending through the downstream end of the center girder, readily accommodates the trunnion hubs of adjacent gates and delivers the water load directly from the gate arms to this girder, thus dispensing with the need of the heavy transverse trunnion girder, as well as with the separate pins and their bearing yokes that are necessary in the conventional anchorage frame.

When the gate loads are unbalanced, as in the case of one gate being relieved of its hydrostatic thrust, the axial stress in the central tie girder is only one-halt of that when the two gates are fully loaded, whereas it was, as we've seen before, one and a half times as large in each side member of the conventional anchorage. The sectional area of the single tension member is still fully utilized, since it is readily proportioned to resist either the total axial load on two gates or the combined axial and flexural stresses due to partial loading. Consequently, no excess metal is required for any loading condition in any portion of the anchorage of the new arrangement. It is also obvious that the outside anchorage of the end gates is identical in design with that of the intermediate gates with the simple exception that the trunnion pin protrudes on one side only of the center girder.

We can reintroduce now the lateral components of the hydrostatic thrust from the inclined gate arms which were deferred for convenience earlier in this presentation. When these components become unbalanced due to unequal gate loading, the resulting lateral force must be transmitted from the steel anchorage to the masonry of the pier. It will be seen by inspection of the drawing that this can be most effectively accomplished by means of a thrust block embedded in the concrete beneath the central tie girder directly under

the pin where the thrust is applied. It should be noted here that the underside location of the thrust block develops torsional stresses in the main girder. However, as in the case of the transverse trunnion girder of the conventional anchorage, the magnitude of the torsional stresses is such as to be of little concern in design and, besides, can be considered to be resisted by the concrete fill placed in the box type central girder after the anchorage has been set in place.

The central tie girder is prevented from bonding to the concrete for the downstream portion of its length in a similar manner as the side members of conventional anchorages. When elongation occurs under stress, the underside of the downstream end slides freely in a fitted groove provided for this purpose in the top of the lateral thrust block.

Figure 3, shows a superposition of the side elevations of the two types of anchorages. It will be seen that at the downstream end both masonry pier and steel anchorage are materially shorter for the novel design than for the conventional type. The anchorage designs shown on this drawing were developed in detail for a project having tainter gates 40 feet wide and 26 feet high. The estimated weights of steel for the gate anchorages of this project are about 18,000 lbs. for each conventional type and only 9,000 lbs. for each single girder anchorage.

The basic arrangement of a central tie girder can be adapted also to tainter gates that must have side arms parallel to and close to the faces of the spillway piers. While the economy of materials and simplicity of fabrication are not as great as in the case of the anchorage with inclined gate arms, the structural merits of a single tie member, single anchor block and single thrust block located along the longitudinal center line of the pier are fully preserved.

The results of a comparative study of the two types of anchorages are summarized in Figs. 4 and 5, which illustrate schematically their fundamental engineering concepts, and in the accompanying table 1 which presents a parallel listing of several of their important features. By inspection of the diagrams and the table, it will be observed that the novel anchorage, as compared to its conventional frame counterpart, results in overall simplification and improvement of structural characteristics, economy of materials, and compact construction with consequent lesser costs of fabrication, handling, and installation.

Table 2, is furnished as a general index of the trend in tainter gate anchorage design as engineered by different agencies for several random projects. Some of the anchorages are essentially bonded to the concrete for the full length of their side members, while others have side members that are free to elongate within the masonry. The indicated weights were estimated from available drawings and are believed to be substantially correct. Attention is invited particularly to column 5 of the table, in which, for purposes of equitable comparison, the weights are given as quantity of anchorage steel per 1,000 lbs. of hydrostatic thrust on the tainter gate. In viewing the anchorage designs listed in this table, in which the several projects are arranged chronologically, and the anchorage designs proper, it is satisfying to observe that, through the years, engineers have accomplished substantial improvement not only in economy of materials but also in the structural functioning of tainter gate anchorages.

Two photographs of installations by the Nashville District further illustrate the fundamental differences between the two types of anchorages. Figure 6

TABLE I - COMPARATIVE DATA

CONVENTIONAL ANCHORAGE

DESIGN FEATURES

1. TWO TRUNNION PINS
2. TWO TRUNNION BASES
3. ONE TRUNNION GIRDER
4. TWO SIDE TIE GIRDERS
5. TWO ANCHOR BLOCKS
6. ONE THRUST BLOCK
7. MASSIVE, RIGID FRAME

FABRICATION, HANDLING AND SHIPPING

8. CUMBERSOME AND EXPENSIVE
9. STRUCTURAL STEEL WEIGHT = 100
10. REINFORCING STEEL WEIGHT AT ANCHORAGE = 100

MASONRY VOLUME AND PLACEMENT

11. VOLUME LARGE, BOTH ABOVE AND BELOW ANCHORAGE
12. RELATIVELY DIFFICULT PLACEMENT ON ACCOUNT OF REINFORCING AND SIDE GIRDERS

COST

13. ESTIMATED TOTAL COST = 100

NEW ANCHORAGE

DESIGN FEATURES

1. ONE TRUNNION PIN
2. NONE
3. NONE
4. ONE CENTER TIE GIRDER
5. SINGLE ANCHOR BLOCK
6. ONE THRUST BLOCK
7. COMPACT, SIMPLE STRUCTURE

FABRICATION, HANDLING AND SHIPPING

8. SIMPLE AND ECONOMICAL
9. STRUCTURAL STEEL WEIGHT = 60
10. REINFORCING STEEL WEIGHT AT ANCHORAGE = 25

MASONRY VOLUME AND PLACEMENT

11. VOLUME MATERIALLY LESS, BOTH ABOVE AND BELOW ANCHORAGE
12. RELATIVELY SIMPLE PLACEMENT AT SIDES AND CENTER GIRDER

COST

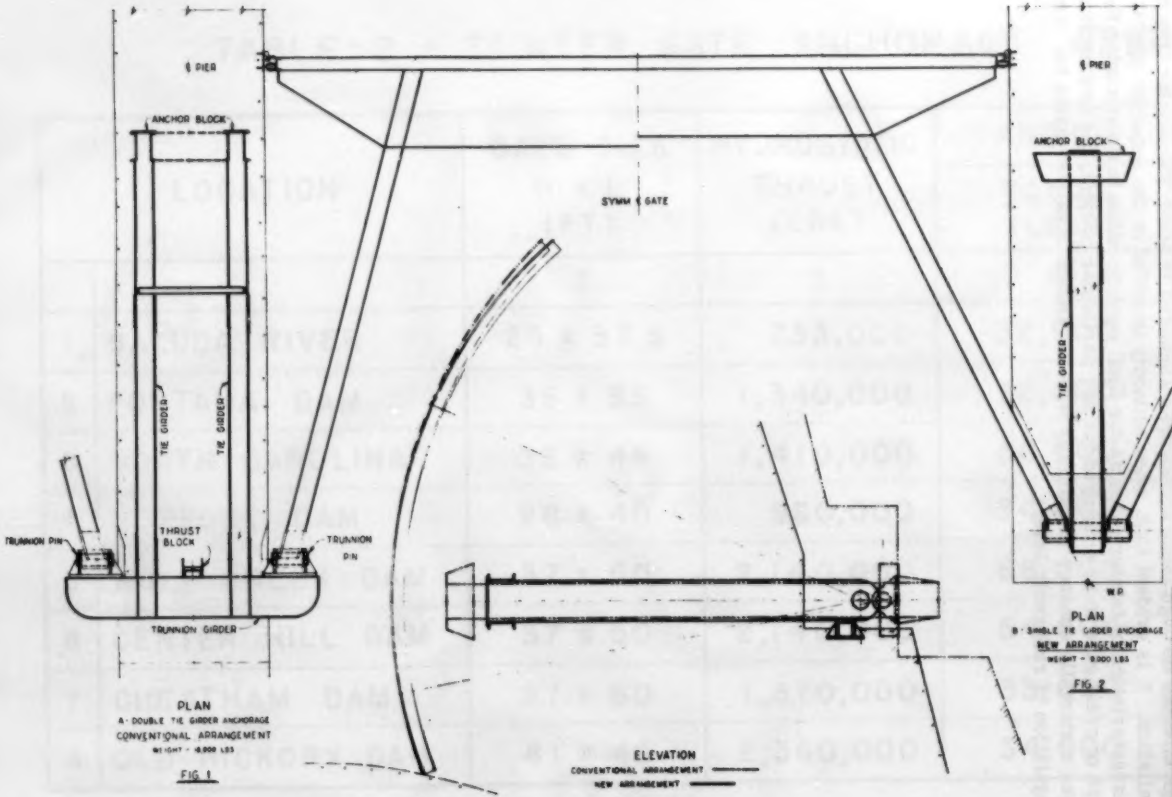
13. ESTIMATED TOTAL COST = 50

TABLE 2 - TAINTER GATE ANCHORAGE WEIGHTS

	LOCATION	GATE SIZE H x L (FT.)	HYDROSTATIC THRUST (LBS.)	ANCHORAGE WEIGHT	
				TOTAL (LBS.)	PER 1000 LBS. OF THRUST
	1	2	3	4	5
1	SALUDA RIVER	25 x 37.5	733,000	32,000	43.6
2	FONTANA DAM	35 x 35	1,340,000	72,000	53.6
3	SOUTH CAROLINA	32 x 44	1,410,000	88,000	62.5
4	NORFORK DAM	28 x 40	980,000	34,000	35.1
5	WOLF CREEK DAM	37 x 50	2,140,000	68,000	31.8
6	CENTER HILL DAM	37 x 50	2,140,000	54,000	25.5
7	CHEATHAM DAM	27 x 60	1,370,000	33,000	24.1
8	OLD HICKORY DAM	41 x 45	2,360,000	34,000	14.4

shows the exposed transverse trunnion girder and the trunnion assembly of the conventional type anchorage for the end tainter gate of the Wolf Creek Dam on the upper Cumberland, while Figure 7 shows the trunnion arrangement of the novel single girder anchorage for two adjacent gates of the Old Hickory project, both as viewed from the tops of the spillway piers.

In closing, it can be stated that the overall merits of the single girder anchorages have now been apparently confirmed by the successful installation and operation of the six tainter gates of the Old Hickory Dam, although they have not been subjected to full head thus far. The estimated monetary savings effected through these anchorages amounted to about $90,000 for this installation. It is evident that, when economy is added to the superior operating characteristics of tainter gates as the final consideration in determining the type of crest gate to be employed for any particular project, the new anchorages materially enhance the already wide advantage of tainter gates for the regulation of reservoir levels.



PLAN
A - DOUBLE TIE GIRDER ANCHORAGE
CONVENTIONAL ARRANGEMENT
WEIGHT - 18,000 LBS

FIG 1

ELEVATION
CONVENTIONAL ARRANGEMENT
NEW ARRANGEMENT

FIG 3

PLAN
B - SINGLE TIE GIRDER ANCHORAGE
NEW ARRANGEMENT
WEIGHT - 9,300 LBS

FIG 2

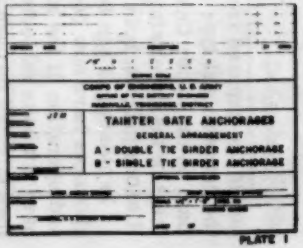

CORPS OF ENGINEERS, U.S. ARMY
TAINTER GATE ANCHORAGES
GENERAL ARRANGEMENT
A - DOUBLE TIE GIRDER ANCHORAGE
B - SINGLE TIE GIRDER ANCHORAGE

PLATE 1

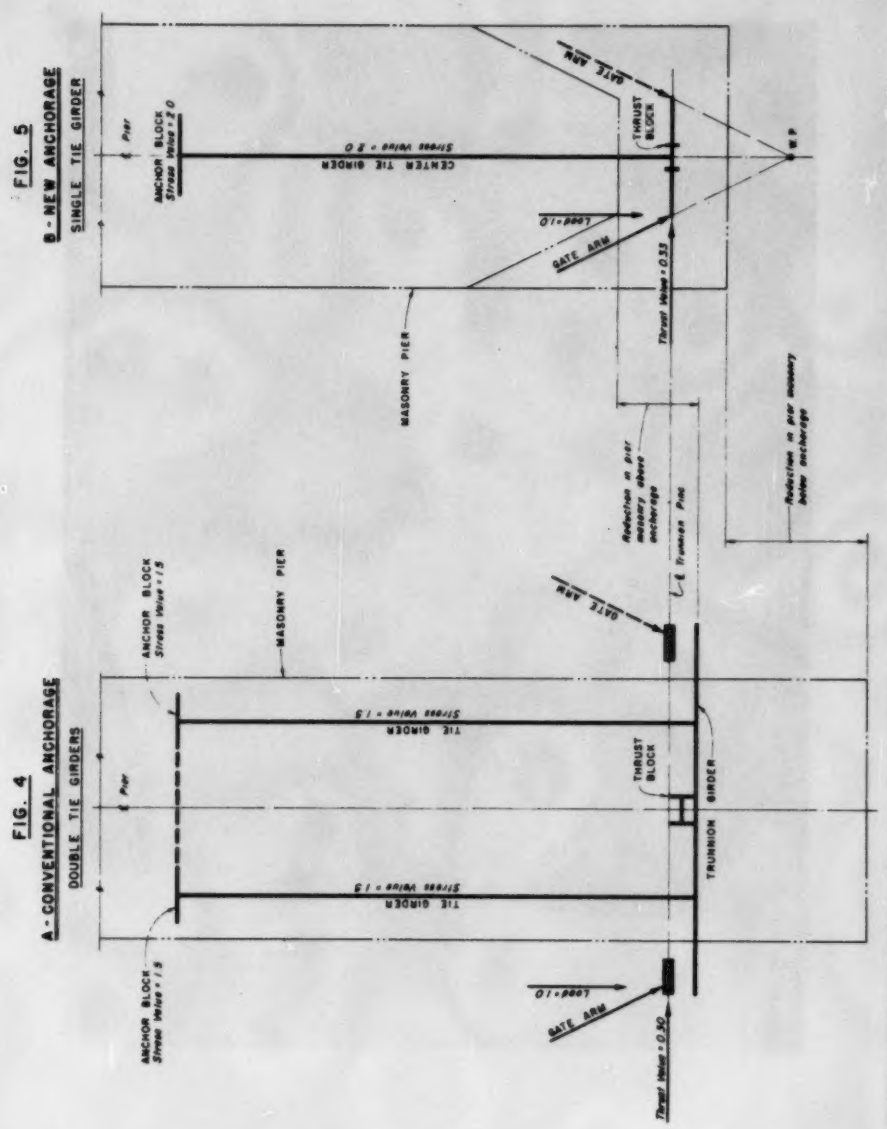


FIG. 4
A - CONVENTIONAL ANCHORAGE
DOUBLE TIE GIRDERS
Pier
ANCHOR BLOCK
Stress Value = 1.5
MASONRY PIER
TIE GIRDER
Stress Value = 1.3
GATE ARM
THRUST BLOCK
TRUNNION GIRDER
Load = 1.0
Thrust Value = 0.50
FIG. 5
B - NEW ANCHORAGE
SINGLE TIE GIRDER
Pier
ANCHOR BLOCK
Stress Value = 2.0
CENTER TIE GIRDER
Stress Value = 2.0
GATE ARM
THRUST BLOCK
Load = 1.0
Thrust Value = 0.33
MASONRY PIER
Reduction in pier masonry above anchorage
Reduction in pier masonry below anchorage

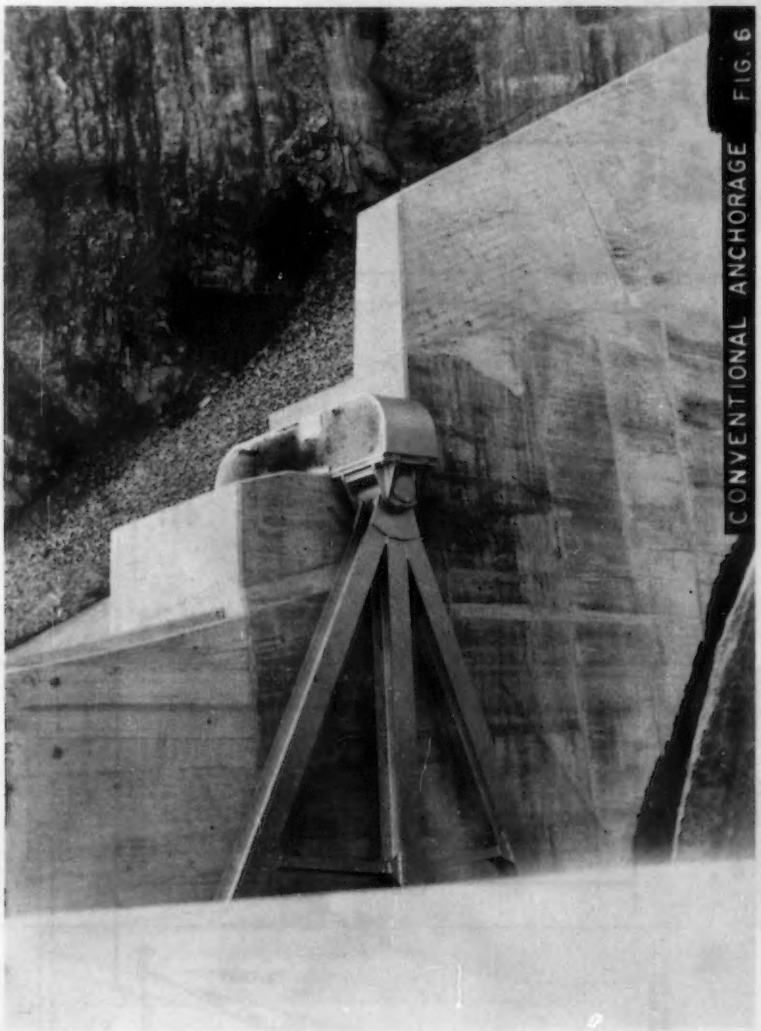
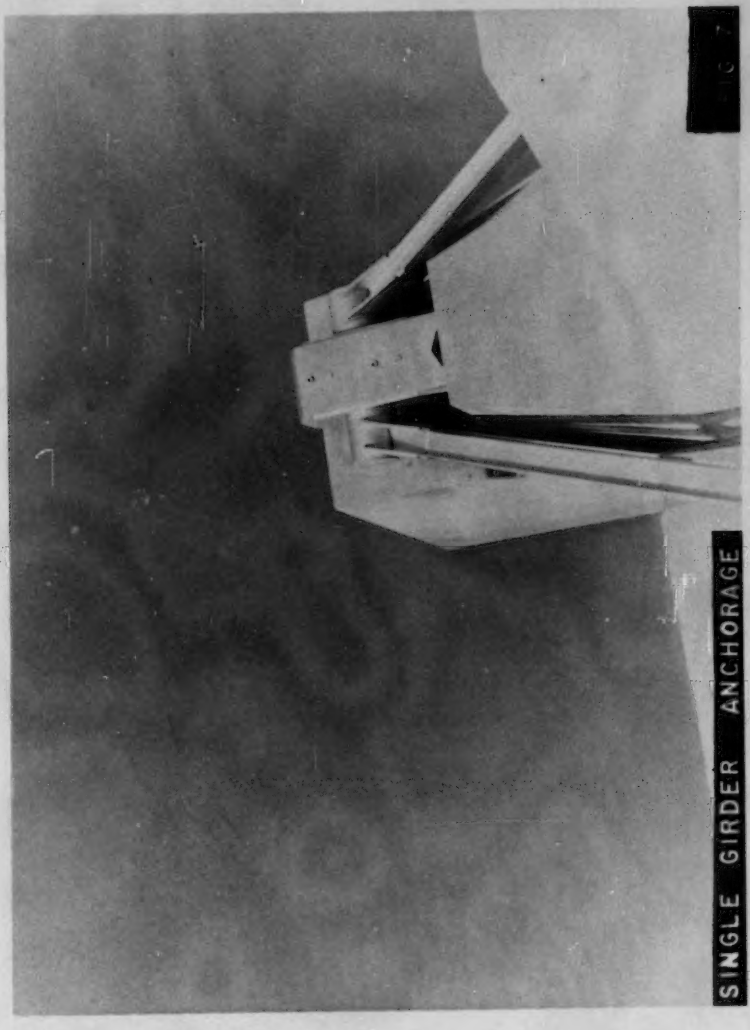


FIG. 6 CONVENTIONAL ANCHORAGE


SINGLE GIRDER ANCHORAGE

Paper 1120 WW 5

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

THE APALACHICOLA RIVER BASIN PROJECT

C. P. Lindner,[1] M. ASCE
(Proc. Paper 1120)

SYNOPSIS

The Federal project in the Apalachicola River Basin in Florida, Georgia, and Alabama is described and discussed in this paper. The primary purposes of the project are navigation, flood control, the production of hydro-electric power, and the regulation of stream flow. Supplementary and incidental benefits are mentioned. Foundations varied from granitoid rock, soft porous and cavernous limestone, and low strength relatively non-porous limestone, to sand and clayey material. In the case of one dam founded on the porous limestone, the problems of water pressures and seepage into the cofferdam were serious and unusual. Unique features and difficult problems encountered during design and construction and their solutions are pointed out and explained.

The Apalachicola River Basin

The Apalachicola River is formed by the confluence of the Flint and Chattahoochee Rivers at the southwest corner of the State of Georgia near the town of Chattahoochee, Florida. From that point it flows along a winding course for about 112 miles to Apalachicola Bay, Florida. In addition to the drainage area directly tributary to the Apalachicola River, its basin includes the drainage areas of the Flint and Chattahoochee Rivers. As may be seen from Figure 1, the uppermost reach of the basin is in northeast Georgia. From there it extends diagonally across the state, encompassing much of the City of Atlanta, thence southward in Alabama and Georgia into Florida. Its total drainage area is about 19,150 square miles of which about 8,500 square miles are tributary to the Flint River and 8,650 square miles are tributary to the Chattahoochee River.

The Flint River rises in the Piedmont Region near Atlanta, flows through

Note: Discussion open until May 1, 1957. Paper 1120 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 5, December, 1956.
1. Chf. Engr., South Atlantic Div., Corps of Engineers, U. S. Dept. of the Army, Atlanta, Ga.

the Pine Mountain Section, an upland area within the Piedmont Region, and traverses the fall line to enter the coastal plain at the latitude of Columbus, Georgia. The Chattahoochee River rises in the Appalachian Mountain Section of Georgia, passes into and through the Piedmont Region to Columbus, Georgia, where it flows over a series of rapids constituting the fall line which forms the boundary between the Piedmont and the Coastal Plain. In the Piedmont and Mountain Regions, the bottom lands generally are narrow, they are flanked by hills or mountains of substantial heights, the stream slopes are relatively steep and the earth mantle is underlain with massive rock formations. The rainfall at the headwaters of the Chattahoochee River averages more than 70 inches a year and the mean annual runoff as far downstream as West Point, Georgia, is in excess of 2 cubic feet per second per square mile of drainage area, a figure more than double the runoff of many eastern streams. As a result of this combination of favorable factors, dam and reservoir sites potentially developable for flood control, stream regulation, and power abound.

In the Coastal Plain region, the stream slopes are comparatively low, and the topography and geology are not favorable to the construction of dams of substantial height. Along the Chattahoochee River, however, the stream has eroded a shallow gorge into the limestone and the indurated earth mantle. This, together with the rolling topography, makes it possible to build low and moderate height dams at locations where the foundation has been found to be adequate for the purpose. The low slope, of course, is favorable for navigation development, for navigation depth can be provided over many miles with a lesser number of structures than would be the case were the slopes steep.

The Federal Project

The existing Federal Project for development of the Apalachicola, Chattahoochee, and Flint Rivers was adopted in the River and Harbor Act of 24 July 1946. This project provides for a flood control, stream flow regulating, and power development at the Buford site north of Atlanta, and a navigation channel 9 feet deep by 100 feet wide from the Gulf Intracoastal Waterway at Apalachicola, Florida, to Bainbridge, Georgia, on the Flint River and to Columbus, Georgia, on the Chattahoochee River to be obtained by stream flow regulation and open channel work in the Apalachicola River and by the construction of three locks and dams at two of which there will be power generating facilities.

The units of the project are shown on Figure 1. In addition to the dam at the Buford site, a lock and dam with power installation known as the Jim Woodruff Dam is located on the Apalachicola River just downstream from the junction of the Flint and Chattahoochee Rivers; a lock and dam without power installation is located at Columbia, Alabama; and a lock and dam with power installation is located at Fort Gaines, Georgia. Major features of these units of the project are shown in Table 1.

The Apalachicola River Navigation Channel

The unimproved channel of the Apalachicola River has a controlling depth at expected minimum flow of 4 or 5 feet. However, many portions have navigable depths equal to or in excess of the 9 feet to be provided by the adopted

project. In many locations the width at that depth is sufficient also. These conditions together with the increased low water flow to be provided by the upstream dams, and reservoirs will permit the 9 foot depth over a 100 foot width with moderate widening at the bends to be obtained without excessive cost by means of dredging. The amount of dredging initially required is estimated to be 2,905,000 cubic yards of which 45,000 cubic yards is rock. Bids for this work were solicited in May 1956.

Buford Dam

As shown on Figure 1, Buford Dam is located on the Chattahoochee River about 30 miles northeast of Atlanta, Georgia. It is a multiple purpose development providing flood control, power, and stream flow regulation. The arrangement in the vicinity of the main dam and a cross-section of the dam are shown on Figure 2. The dam which has a maximum height of about 200 feet consists of three saddle dikes and the main dam, all of which are constructed of rolled earth fill with an upstream mantle of rock. Forebay and tailrace channels were excavated into the hill beyond the right bank of the stream to maximum depths of almost 200 feet, 100 feet of which was in earth and 100 feet in solid granitoid rock. Three main tunnels were driven through the rock to connect the channels. Two of the main tunnels, each 22 feet in inside diameter and lined with concrete and steel, will serve as penstocks. A tunnel, 10 feet in diameter similarly lined, branches from one of the main penstock tunnels. It will serve the comparatively small power unit, [see Table 1]. The third main tunnel, 13.25 feet in inside diameter, is the flood control sluice. It is lined with concrete only as its control is at the upstream end so it will not be subjected to full headwater pressure. Since the forebay and tailrace channels and the tunnels were used for diversion of the river during construction of the main dam, it was necessary to excavate the forebay essentially to river level and to locate the tunnels at about that elevation also.

The headworks were constructed against the rock face at the lower end of the forebay channel. Contained therein are the racks, the head gates and the sluice gates. The gate operating machinery surmounts the headworks.

The powerhouse is under construction adjacent to the vertical rock face at the head of the tailrace cut. It will contain two 40,000 kilowatt units and one 6,000 kilowatt unit driven by Francis turbines operating at an average head of 140 feet. The average annual output of electrical energy will be about 170,500,000 kilowatt hours. The 6,000 kilowatt unit will utilize flows that must be permitted to pass the dam to care for municipal and industrial needs downstream.

Several features of the powerhouse and its intake and discharge channels are worthy of note. The powerhouse is nestled in a deep rock cut. This open rock cut and the forebay rock cut are about 100 feet deep, with the total depths of these excavations including the overburden approaching 200 feet. The depth of these open cuts was dictated by an economic study. It was found that the cost of tunneling plus surge tanks which would have been required with longer tunnels exceeded the cost of open cut excavation. There are no steel penstocks standing free within the penstock tunnels. The steel serves simply as a water-tight liner. During construction it was used as the form behind which concrete was pumped to fill the space between the steel and the rock. By installing the steel against rigid backing in this manner, and using the

stiffeners that were needed for rigidity during erection and the placement of concrete, to tie the steel to the concrete, the thickness of the steel liners could be reduced except near the downstream portals of the tunnels.

The unusual feature of the plan is that there is no concrete whatever in any part of the dam, the saddle dikes, and the spillway. Had a lower dam been built, an expensive concrete spillway would have been required. By raising the dam and thereby providing more flood storage it was practicable to excavate a relatively small channel through the rock ridge to serve as the spillway. Because of this, the higher dam was less expensive than a lower dam originally proposed, having essentially the same power capacity and a reasonably adequate amount of flood storage.

The cross-section on Figure 2 shows the design of the dam. It consists principally of selected rolled earth fill with a heavy blanket of rock on the upstream face. A core trench ties the relatively impervious earth section to the foundation rock which was grouted to minimize leakage. Under the downstream third of the dam, a drainage blanket has been placed. This conducts seepage to the rock toe which extends above tailwater and thereby protects the dam from wave wash from that source. All of the rock used in the dam was obtained from excavations made for other features, principally from the forebay and tailrace cuts. Total quantity of material in the main dam which was completed in 11 months is about 4,000,000 cubic yards. About an additional million cubic yards is contained in the saddle dikes.

The reservoir clearing is worthy of mention because the provisions are not common and may be used elsewhere. The specifications required complete clearing down to elevation 1030 feet m.s.l., 5 feet below the minimum power pool, and a clear pool at that elevation. Since no trees, branches, or projections will be permitted to protrude above elevation 1030 feet m.s.l., the contractor has chosen to clear completely by land methods to an elevation somewhat lower. This overclearing is optional with the contractor and the extent depends on his estimate of the comparative costs of clearing by land methods and by floating plant. Between the lower limits of complete clearing the contractor will top the trees and branches at elevation 1030 ft. m.s.l. or lower by the use of floating equipment.

Effect of Buford Dam and Reservoir

Buford functions in conjunction with the remainder of the project below Columbus, Georgia, by increasing the low water flow for power production and for lockages at the downstream sites, and for maintaining navigable depths in the Apalachicola River. Its regulation is utilized in the design of the navigation channel in that stream and as a result less initial dredging is required to obtain the desired low water cross-section. In addition, high tailwater during floods reduces the hydroelectric capacity at Fort Gaines and Jim Woodruff powerhouses. At such times a substantial proportion of the loss can be replaced by Buford for its capability at that time will be beyond its low water period dependable capacity and, in addition, it can be operated at overload. It can be seen therefore that Buford is a fundamental part of the remainder of the project below Columbus, Ga.

Buford Reservoir is capable of completely controlling floods at the site. Its firm flood storage capacity is equal to twice the volume of the greatest flood series of record at that point. It will have a substantial effect on many

floods as far downstream as West Point, Georgia. At Atlanta, with but a few exceptions, all floods will be restricted to bankful stage. Table 2 shows the control that Buford could have exercised on past floods. The January, 1946, stage was the highest of record at Vinings [Atlanta]. That flood could have been reduced to a no-damage stage by Buford. It should be noted that even with Buford operating, a few past floods would have exceeded flood stage slightly. In fact, on March 16, 1956, with only about 500 cubic feet a second being released from Buford Dam, a discharge of 15,100 cubic feet a second and a stage of 12.1 feet were recorded at Vinings. When it is remembered that Buford is a main stem reservoir a little less than 50 river miles from Vinings, the difficulty or impracticability of providing a substantial degree of downstream flood control on a main river by means of small headwater storage or detention works may be realized.

The increase in low water flow accomplished by Buford Reservoir is important and of benefit not alone to the Federal project. Georgia Power Company Dams, several of which are located between Buford and Columbus, Georgia, will be aided. But more important, perhaps, is the insurance it gives to industries and communities downstream, including the City of Atlanta, Georgia, that they will henceforth have a dependable water supply. It is planned that the minimum amount of water to be released from Buford will guarantee 600 cubic feet a second at Atlanta. But the reservoir will regulate the stream flow to an average flow during low water seasons of about 1600 cubic feet per second. With a modest amount of supplementary work, the communities and industries can assure themselves of a steady flow of at least that amount during low water seasons.

Fort Gaines Lock and Dam

Fort Gaines Lock and Dam is a multiple purpose development which will function principally for navigation and the production of hydroelectric power although the reservoir will be drawn down prior to the flood season and as a result a measure of flood control will be provided. It is located on the Chattahoochee River about 85 river miles downstream from Columbus, Georgia, near the community of Fort Gaines, Georgia [see Fig. 1]. The dam will consist of earth abutments 11,940 feet long flanking a center section containing the powerhouse, the gated spillway, the non-overflow sections and the lock. The arrangement and approximate lengths of individual sections are shown on Figure 3.

The powerhouse will contain 4 units of 32,500 kw capacity each for a total capacity of 130,000 kw. The generators will be driven by high specific speed, Kaplan type turbine runners. The estimated average annual output of the plant will be 436,000,000 kwh. The operating net head at full load will be about 73 feet. During high floods the operating head will be reduced materially by tailwater rise. At such times the reduced output will be compensated by operation at higher than dependable capacity and, if necessary, at overload, of plants, such as Clark Hill, Buford, Allatoona, and others that are not penalized by tailwater rise and have an excess of water when floods occur on the Chattahoochee River. As has been stated previously, the river channel for a great distance downstream is gorge like, since the banks are vertical or almost so. This, together with the low natural slope, accounts for the tailwater rise. For moderate increases in discharge, the rise is such as to require

control of the rate at which the power units can be placed in operation and brought to full load. Otherwise, the wave that would be created would in its progress downstream constitute a hazard to navigation.

Cross sections of the gated spillway and of the west embankment are shown on Figure 3. Materials underlying these sections, typical of the site, are also shown. Over the valley floor, the general height of the earth sections of the dam will be about 60 feet. Immediately below the surface under the embankment section shown and upstream and downstream therefrom there is a relatively impervious blanket of varying thickness, but generally 5 feet or more deep. This is underlain by sand 20 feet or more in depth extending to the limestone below. Because of the existence of the impervious blanket, it was decided to utilize it to reduce seepage to acceptable limits. Therefore, the impervious blanket is to be retained and strengthened by impervious fill where necessary to insure a blanket at least 5 feet thick extending 500 feet upstream from the toe of the upstream slope of the dam. Under the downstream 1/3 of the earth section, a select pervious drainage fill has been provided terminating in a gravel toe. Should toe drains be found necessary, they will be installed later. A drainage ditch will conduct the seepage to the river.

The concrete structures, [see Figure 3], are founded on rock. Two principal rock formations underlay the site. The rock nearest the surface has been designated earthy limestone. Below that is a formation termed shell limestone. The earthy limestone is massive, without bedding planes, and is relatively impervious. So far as can be ascertained it is free of solution channels. Tests indicate that its average unconfined compressive strength is about 21 tons per square foot. There is, however, a considerable variation in individual test results. The shell limestone is also massive and without bedding planes. It is filled with shell fragments and is considerably more porous than the earthy limestone although it does not appear to have large solution channels. It is much stronger than the earthy limestone. Tests made show that its average unconfined compressive strength is about 35 tons per square foot.

The spillway and a portion of the non-overflow section of the concrete dam are to be founded principally on the earthy limestone. In order to obtain a satisfactory factor of safety against sliding it was found necessary to key these sections into the shell limestone. This will be done by generally sloping the base of the dam downward from the toe toward the heel of the dam. In addition, the shear plane will be further projected and lengthened in the shell limestone by anchoring the stilling basin into that formation as shown on Figure 3. Both the powerhouse and the lock will be founded on the shell limestone. The depth of these structures required that they extend to this formation, but, especially in the case of the lock, the use of the shell limestone appeared judicious under any condition as the maximum lift is 88 feet and the walls are about 130 feet high.

Results to be Obtained from the Fort Gaines Development

The Fort Gaines development will extend navigation with a minimum depth of 9 feet from the pool created by the Columbia Lock and Dam to the city of Columbus, Georgia. The lock, which will have clear dimensions of 82 by 450 feet, will permit passage of substantial tows without breaking of the tows for multiple lockages. A substantial block of hydroelectric power will be produced

in an area where it has been difficult in recent years to maintain a margin between capacity and demand. The reservoir will be drawn down for the flood season and as a result most floods will be reduced modestly. An interesting fact regarding this drawdown is that by its use the cost of reservoir relocations and land will be reduced—in effect, upstream as well as downstream flood control is accomplished. As the area of the reservoir at elevation 190 feet, m.s.l. will be 46,000 acres, an excellent facility will be provided for recreational development and enjoyment.

Columbia Lock and Dam

Columbia Lock and Dam is located on the Chattahoochee River, about 29 miles downstream from Fort Gaines Lock and Dam, near the community of Columbia, Alabama. It will consist of a low concrete dam, with a lock constructed in the left river bank. No power installation and no flood control storage are provided. The major portion of the dam will be an uncontrolled spillway. One or two spillway gates will be provided to permit drawing down of the pool prior to the delivery of water from the Ft. Gaines turbines. Although the exact design has not yet [June 1956] been established, the general layout and a cross-section of the uncontrolled spillway are shown on Figure 4. The minimum full upper pool elevation will be about 102 feet m.s.l. The maximum height of the dam from the foundation to the crest of the uncontrolled spillway will be about 62 feet. The maximum lift of the lock will be 25 feet. This will occur with the Jim Woodruff pool below at its static elevation of 77 feet m.s.l. When the discharge exceeds about 50,000 cubic feet per second, the uncontrolled spillway will be submerged by the tailwater. For discharges in excess of about 100,000 cubic feet per second, vessels can pass over the dam.

Subsurface investigations have shown that there is multiple interbedding of formations at the site of limestone, sandstone, claystone, and occasional clay with sand under artesian pressure below about elevation 15 feet m.s.l. Although the foundation is not ideal it appears adequate for the moderate heights and weights of structures to be placed upon it.

Results to be Obtained from Columbia Lock and Dam

This lock and dam will extend the navigation channel with a minimum depth of 9 feet from the Jim Woodruff pool below to Fort Gaines Lock and Dam upstream. The lock will have a clear chamber 82 feet wide by 450 feet long which is sufficient to permit the sized tows expected to use the project to pass through the lock in one lockage. By drawing down the Columbia pool in advance of the turbine discharge at the Ft. Gaines development, these discharges will be reregulated to reduce the rate of rise in the upper end of the Jim Woodruff pool and thereby minimize hazards to navigation. By proper operation the rate of rise in a portion of the pool upstream from Columbia can be reduced also. This operation was found to be desirable because of the channel characteristics which result in high increments of stage per increment of discharge change.

Jim Woodruff Lock and Dam

Jim Woodruff, like Fort Gaines, is a multiple purpose development. It will provide for navigation, produce hydroelectric power, and will reregulate flows for the open river navigation channel downstream. It is located on the Apalachicola River which is formed by the confluence of the Flint and Chattahoochee Rivers about 1/2 mile above the dam. The dam will back water about 47 river miles to Columbia Lock and Dam on the Chattahoochee River and about 47 river miles up the Flint River to a point about 18 miles upstream from Bainbridge, Georgia. The dam site crosses the Georgia-Florida line near the southwest corner of Georgia.

Figure 5 shows the layout at the dam site. The development, proceeding from east to west, consists of an overflow earth dike protected both upstream and downstream by grouted riprap, an elevated switchyard earth section, a powerhouse, a gated spillway containing 16 double leaf vertical lift gates, a lock, and an ungated spillway. The total length of the structure is about 6,130 feet. The maximum height of the dam from the foundation to the gated spillway operating deck, elevation 107 feet m.s.l., is about 100 feet. Normal pool elevation is 77 feet m.s.l. The crest of the ungated spillway is at elevation 79 feet m.s.l. and the top of the overflow dike is at elevation 85 feet m.s.l. The maximum lift of the lock of 33 feet occurs with minimum tailwater of 44 feet m.s.l. and normal pool elevation. Except in time of high water, flows will be passed through the power units. As flows increase beyond the capacity of the turbines, the pool will be held at elevation 77 feet m.s.l. by operation of the gated spillway until its capacity is exceeded. When the pool rises above elevation 79 feet m.s.l., water will discharge over the ungated spillway which has a clear length at that elevation of 1,584 feet. During extreme floods the pool and tailwater will rise so when the overflow dike on the east bank is overtopped, the swell head will be slightly less than 2 feet. Navigation will have been interrupted slightly prior to that time.

Three 10,000 kw generators driven by Kaplan type runners are being installed in the powerhouse. They will produce an average annual energy output of 212,000,000 kwh. As implied above, the tailwater rise is such that during many floods the capacity of the units will be reduced and during high floods they will be unable to operate. At such times the lost capacity will be replaced by operation of Buford and other plants where an excess of capability will be available beyond assigned dependable capacities. The Jim Woodruff power units will normally be operated on a 24-hour basis in order that sufficient water will be discharged continuously to maintain a 9 foot depth for navigation in the Apalachicola River Channel downstream. Since Fort Gaines power plant upstream will not operate continuously, this operation at Jim Woodruff will regulate and regularize the varying flows from above. During low water season, especially on week ends this may result in drawing down of the Jim Woodruff pool one or two feet. As the pool area at normal pool elevation is 37,500 acres, a substantial pondage is available for this purpose in the top 2 feet.

The geologic formations underlying the structure shown on the cross-section presented on Figure 5 are typical of the site from the west abutment to and including the powerhouse. The rock is Tampa limestone. It is generally a soft, porous stone, containing numerous solution cavaties and channels. However, the stratum designated on the cross-section as the "D" zone is comparatively impervious with respect to the other layers. The stone below

the "D" zone has the highest permeability. It is apparent that a deep grout curtain was required under the dam and this has been provided except under the switchyard fill east of the powerhouse. The "D' zone does not extend across this area and the surface of the rock is deeply depressed in what appears to be an ancient channel. This channel is filled with sand and gravel which is quite permeable. To control the seepage along this path a sheet pile cutoff has been driven to rock from the wing wall of the powerhouse across this old channel, and drainage wells have been provided at the downstream toe of the fill. Eastward of this under the overflow dike, a grout curtain has been installed.

Becuase of the high permeability of the underlying formations, cofferdam grouting was used to reduce seepage into the work areas. Despite this, cofferdam pumping has been enormous. By far, the worst phase from this standpoint was the last stage cofferdam. It inclosed a part of the gated spillway, the powerhouse, and the ancient channel mentioned above. For many months, 15 to 18 million gallons of water a day were pumped from this cofferdam with a high daily rate of 20 million gallons.

An interesting feature of this project is the lack of drainage wells downstream from the grout curtain under the concrete dam. Because the tailwater elevation is above the grouting and inspection gallery, it was impracticable to install drainage wells below the grout curtain. To have done so would have enabled water from the tailwater to have passed in great quantities through the porous rock and the drainage wells into the gallery. As a result the dam must withstand full headwater pressure to the grout curtain and full tailwater pressure or greater downstream from the grout curtain. It was checked for stability with full pressure on the base varying uniformly from headwater to tailwater.

Another unique problem was encountered with respect to draft tube drains under the powerhouse. It was found that full tailwater pressure existed in the rock under the "D" zone and that large quantities of water would flow through drain holes extended through that zone. Above the "D" zone the pressure could be reduced to acceptable intensities by weep holes and the seepage to be expected could reasonably be pumped when the draft tube was unwatered. It was also found that if the pressure below that zone were transmitted above the "D" zone through drain holes, the integrity of the floor might be endangered under many tailwater heights. As a result, it was decided to provide shallow drains under the draft tube slabs, and to install drains through the "D" zone only downstream from the powerhouse. The purpose of the latter drains is to prevent headwater and artesian pressure that may be present in excess of tailwater pressure from reaching the area under the draft tubes.

Results to be Obtained from Jim Woodruff Lock and Dam

Jim Woodruff development will provide a minimum depth of 9 feet for navigation from the Apalachicola River Channel improvement below to Bainbridge, Georgia, on the Flint River and to Columbia Lock and Dam on the Chattahoochee River. The lock will be of the same clear dimensions as the other locks in the project, so it will be able to pass expected tows in a single lockage. The pool will regulate the varying flows from Fort Gaines Development during low water seasons so as to provide a reasonably constant flow of sufficient amount to maintain 9 foot navigable depths in the Apalachicola River

channel downstream. The power plant will produce an average of 212,000,000 kwh of electric energy annually. Since it will operate on a 24-hour basis it will be one of the few hydroelectric plants to operate on base load during low water seasons that have been constructed in recent years. In addition, the 37,500 acre lake will offer water recreational opportunity unparalleled in most localities, especially for fishing and for hunting since the lake should provide natural habitat for waterfowl.

Status of the Project and Expected Completion Dates

As of June 1956, improvement of the Apalachicola River Channel is about to begin. Completion in the fiscal year 1957 is expected.

All major construction items of the Buford Development have been completed except the powerhouse and switchyard. In addition, several relocations are still under construction and a few that are required have not been started. Reservoir clearing is underway. It will be completed by topping out by floating plant methods. The reservoir has been filled to an elevation in excess of 1000 feet m.s.l. It is hoped that construction of relocations will progress with sufficient rapidity to permit the reservoir to be filled to at least elevation 1035 feet m.s.l., the minimum power pool elevation, by the time the first power unit is ready to operate which is scheduled for April 1957. The entire development will be completed by August 1957.

Construction began on the Fort Gaines development in October 1955 when the contractor for the west embankment started earth moving operations. Since that time contracts have been awarded for construction of the first stage cofferdam, and for the furnishing of steel for that cofferdam. The work will continue on the development by the awarding of additional contracts at rates consistent with the completion of design and with the appropriations made. The powerhouse will be the last major element of the project to be completed. The first power unit will begin operation about the first part of 1962, and the entire development is scheduled for completion during that year.

Construction of Columbia Lock and Dam is expected to start in 1959 or 1960. It will be completed essentially at the same time as Fort Gaines, i.e. in 1962. Since this lock and dam involve relatively small quantities, a long construction time is not needed.

Work is underway on the last stage of construction of the Jim Woodruff Development. This stage includes the final portion of the gated spillway, the powerhouse, and extension of the earth fill on the east bank to connect with the powerhouse intake section of the dam. The first power unit will be placed in operation in December 1956 and the entire development will be completed about April 1957.

It should be noted that Buford Dam and Reservoir are operating now to regulate stream flows and control floods and that hydroelectric power will be produced at Jim Woodruff Dam, starting in December 1956 and at Buford Dam, starting in April 1957. Another primary purpose of the project soon to be realized is 9 foot depth navigation from the intracoastal waterway to Bainbridge, Georgia, on the Flint River and Columbia, Alabama, on the Chattahoochee River. This navigation channel will be available when the Apalachicola Channel improvement is completed early in 1957. Following that, both Columbia Lock and Dam and Fort Gaines Development will be completed about the same time, so that by the end of 1962 the entire project will be completed and all project purposes will be served.

SUMMARY AND CONCLUSIONS

The Federal Project for the Apalachicola River Basin consists of channel improvement of the Apalachicola River; 3 locks and dams, one near the head of the Apalachicola River to contain a power installation, and two on the Chattahoochee River below Columbus, Georgia, one of which will contain power generating facilities; and a multiple purpose dam and reservoir on the Chattahoochee River above Atlanta, Georgia, for flood control, stream flow regulation, and the generation of hydroelectric power. The entire project is scheduled for completion in 1962, although some of the project purposes will be served prior to that time. In fact, stream flow regulation and flood control in the Atlanta area are a reality at the present time as is flood reduction at points farther downstream. When the project is completed, in addition to the flood control and stream flow regulation, it will provide a navigation channel with minimum dimensions 9 feet deep by 100 feet wide from the intracoastal waterway at Apalachicola, Florida, to Bainbridge, Georgia, on the Flint River, and to Columbus, Georgia, on the Chattahoochee River. The project hydroelectric power generating facilities will have a total capacity of 246,000 kw and will produce an average of 819,000,000 kwh annually.

The three large reservoirs created will offer unexcelled opportunities for water sports and recreation of all types. The Buford Reservoir which is very close to the Atlanta metropolitan area will be intensely used for recreational purposes. Besides providing the usual recreational opportunities, Fort Gaines and Jim Woodruff show promise of furnishing excellent havens for waterfowl, so hunting may be a popular sport on these reservoirs.

In its function of regulating and conserving water supply for productive use, the project will be of extreme value to the area served. Industries, farms, and communities, including Atlanta, Georgia, will be assured of an abundant and dependable water supply. Many modern industries require a large amount of water for cooling and processing purposes. The availability of the supply created by the project should attract industries because of the economy of locating near an adequate source of water. Industry also seeks low cost transportation and a satisfactory supply of power at reasonable rates, both of which will be promoted by the project. Insofar as the project will enable goods to be produced and transported at less cost than elsewhere, will protect crops and improvements from damage and destruction, and will enable equal or better crops to be raised with less effort thereby conserving energies for additional economic activities, it will serve the economy and the welfare of the entire nation.

Table 1.

DATA ON FEDERAL PROJECT, APPALACHICOLA RIVER BASIN

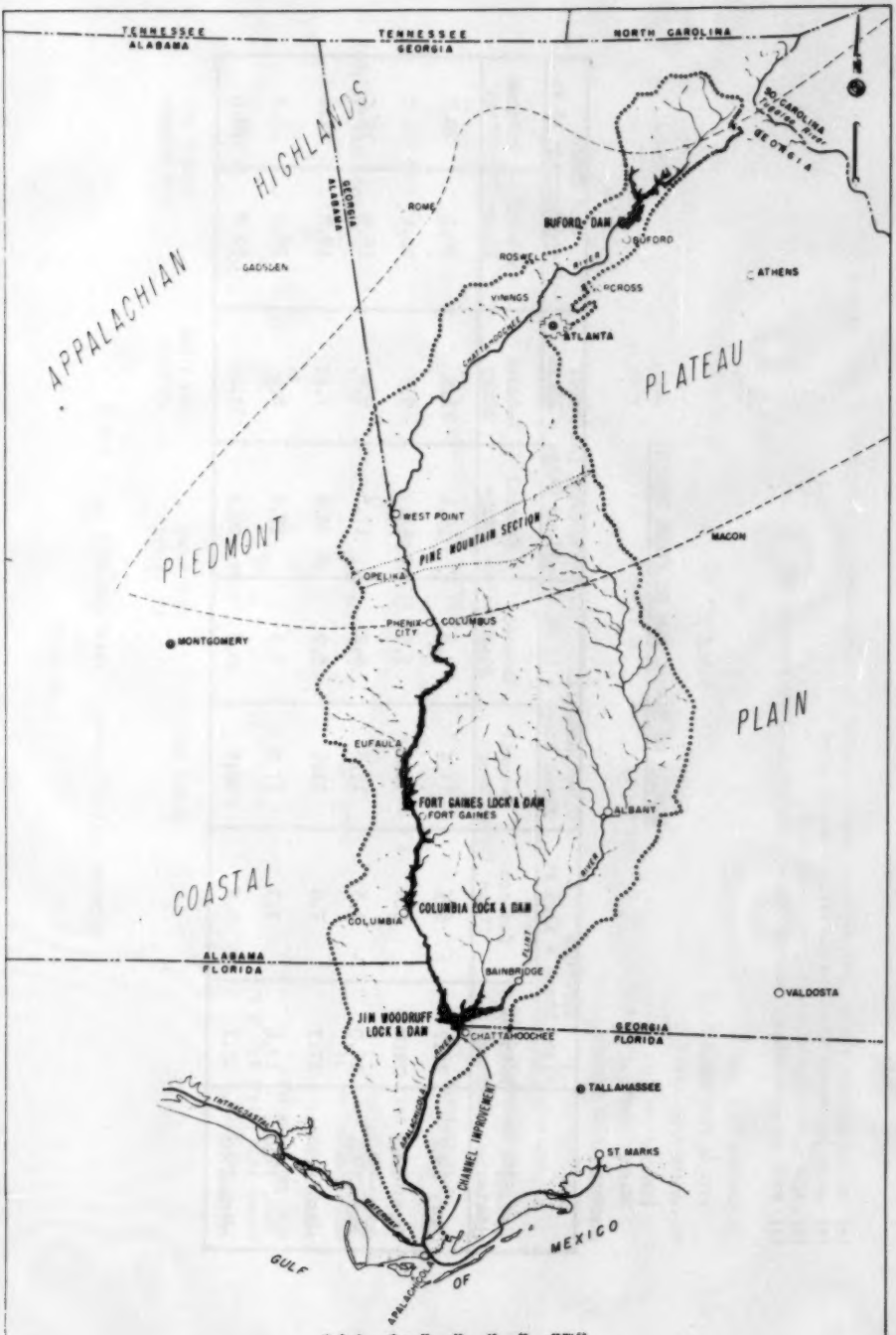
	BUFORD DAM	FT. GAINES LOCK & DAM	COLUMBIA LOCK & DAM	JIM WOODRUFF LOCK & DAM
DRAINAGE AREA, sq. mi.	1,046	7,507	8,213	17,150
NORMAL POOL ELEV., ft., m.s.l.	1,070	190	102	77
MIN. TAILWATER ELEV., ft., m.s.l.	915	102	77	44
STATIC HEAD, ft.	155	88	25	33
POWER STORAGE, ac. ft.	1,049,000	210,000(1)	None	Pondage only (37,000)
GENERATING UNITS ----				
NUMBER & UNIT CAPACITY	2 @ 40,000 kw 1 @ 6,000 kw	4 @ 32,500 kw	None	3 @ 10,000 kw
TOTAL CAPACITY	86,000 kw	130,000 kw	None	30,000 kw
AVE. ANNUAL OUTPUT, kwh.	170,500,000	436,000,000	None	212,000,000
FIRM FLOOD CONTROL STORAGE ----				
BETWEEN ELEV., ft. m.s.l.	1070 & 1085	185 & 190(2)	None	None
QUANTITY, ac. ft.	637,000	210,000(2)	None	None
SURCHARGE FLOOD STORAGE(3)----				
BETWEEN ELEV., ft. m.s.l.	1085 & 1100	190 & 206.9	None(4)	None(4)
QUANTITY, ac. ft.	780,000	975,000	None(4)	None(4)
NAVIGATION LOCK ----	None			
SIZE OF LOCK CHAMBER, ft.	-	82 by 450	82 by 450	82 by 450
MAXIMUM LIFT, ft.	-	88	25	33

(1) Used for flood control storage during flood season -- see footnote (2).
(2) Firm flood storage during flood season only.
(3) Range and amount given are for spillway design flood.
(4) During spillway design flood and other extreme floods, the river stages are unaffected by the structure.

TABLE 2

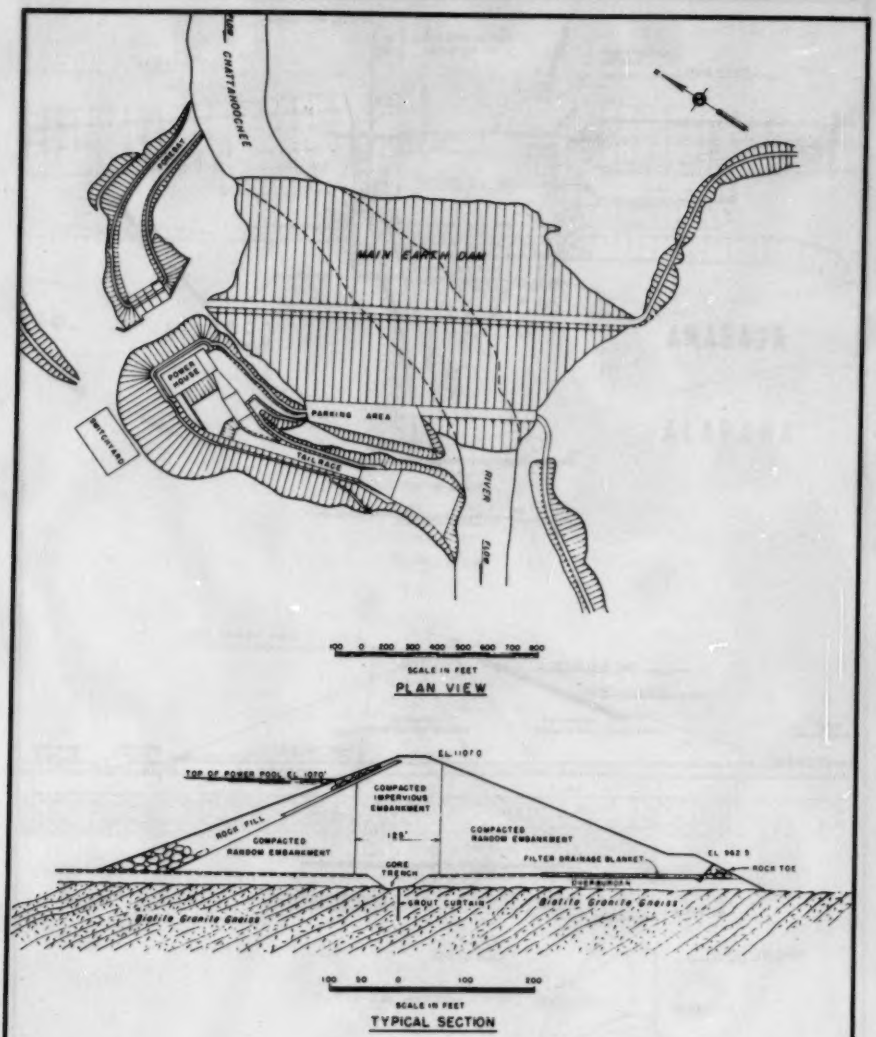
EFFECT OF BUFORD DAM ON FLOOD STAGES

Flood Date	Norcross Flood Stage = 11.5 Ft		Roswell Flood Stage = 11 Ft		Vinings [Atlanta] Flood Stage = 11 Ft		West Point Flood Stage = 16 Ft	
	Natural Stage	Reduced Stage	Natural Stage	Reduced Stage	Natural Stage	Reduced Stage	Natural Stage	Reduced Stage
Mar. 1942	15.2	6.5	13.8	7.8	15.1	11.5	20.1	20.1
Mar. 1943	11.8	3.8	11.7	5.9	13.7	9.3	20.4	20.0
Mar. 1944	18.0	3.5	15.8	6.2	17.6	9.0	16.0	15.5
Jan. 1946	27.7	5.0	23.4	6.2	28.0	7.6	19.8	17.6
Feb. 1946	19.4	3.2	17.0	6.3	19.2	8.4	15.6	12.3
Jan. 1947	21.1	9.5	17.8	10.8	20.1	13.2	19.8	19.0



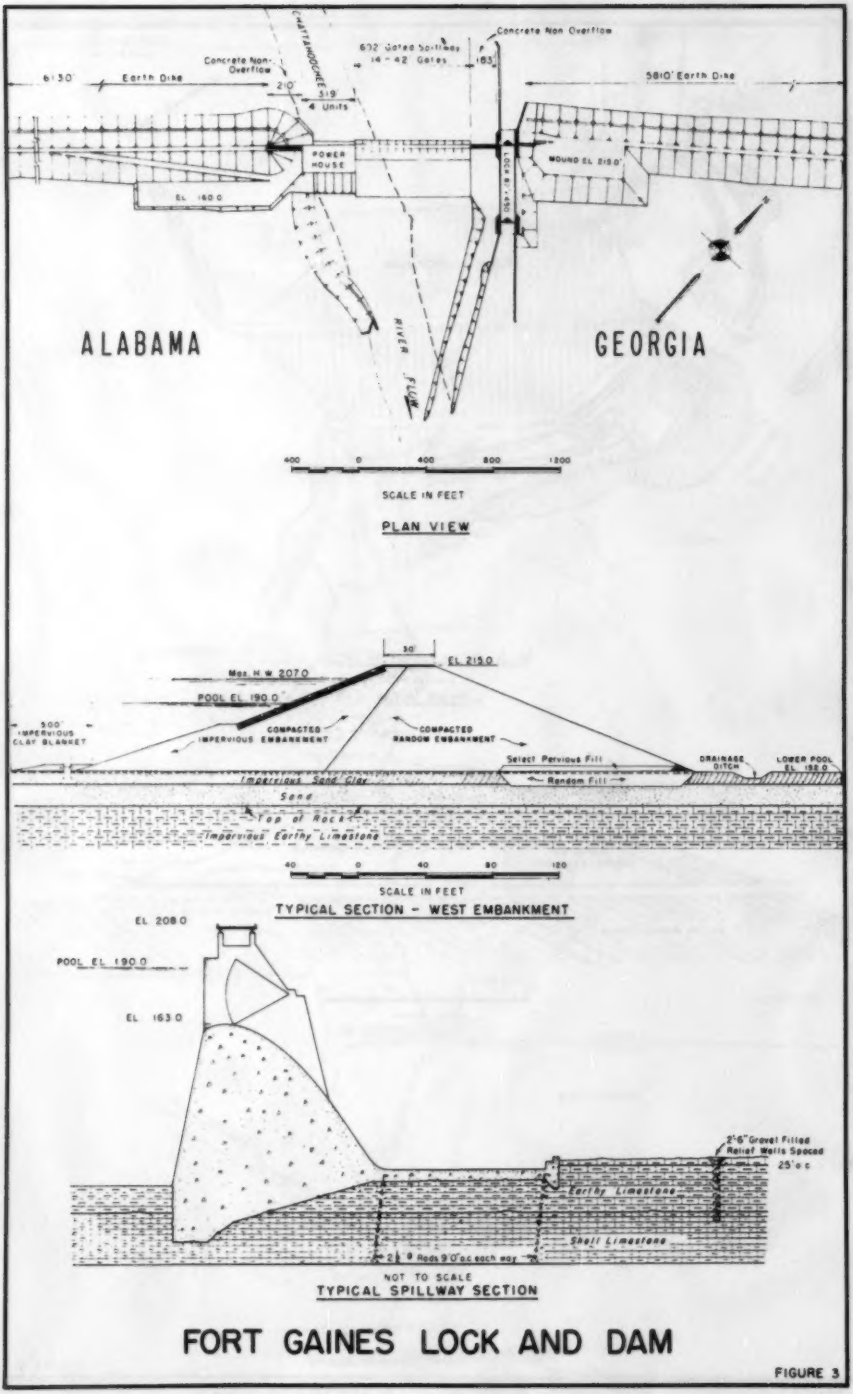
APALACHICOLA RIVER BASIN

FIGURE 1



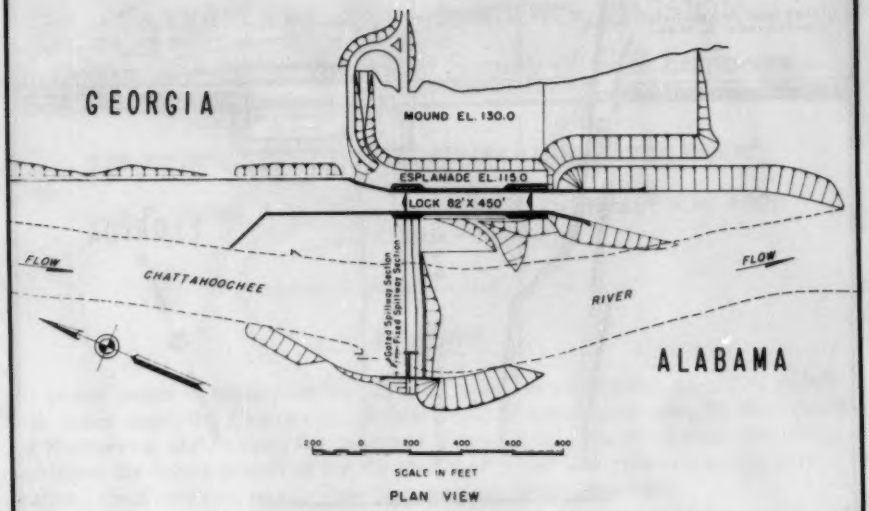
BUFORD DAM

FIGURE 2

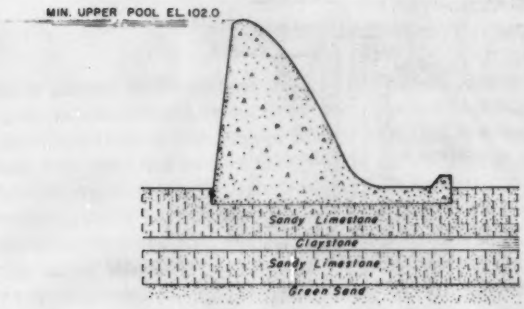


FORT GAINES LOCK AND DAM

FIGURE 3



PLAN VIEW

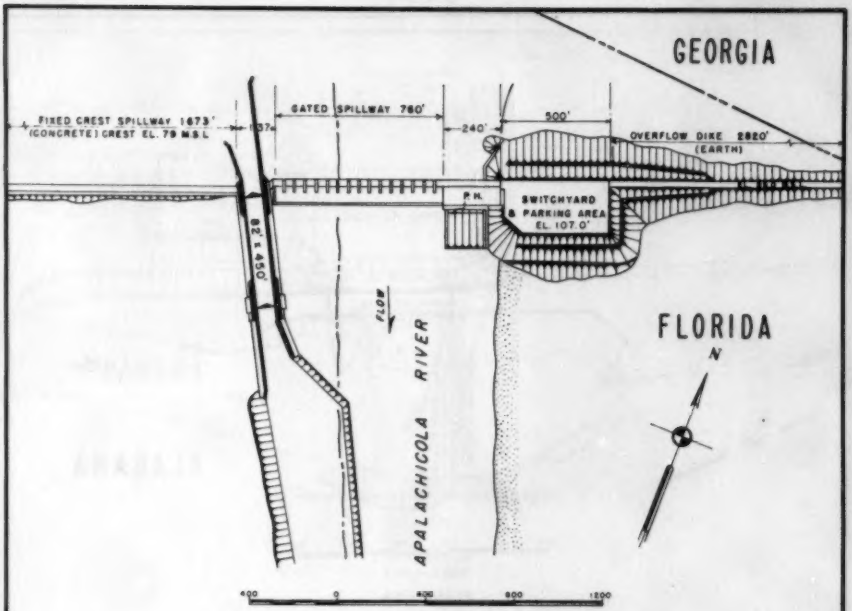


NOT TO SCALE

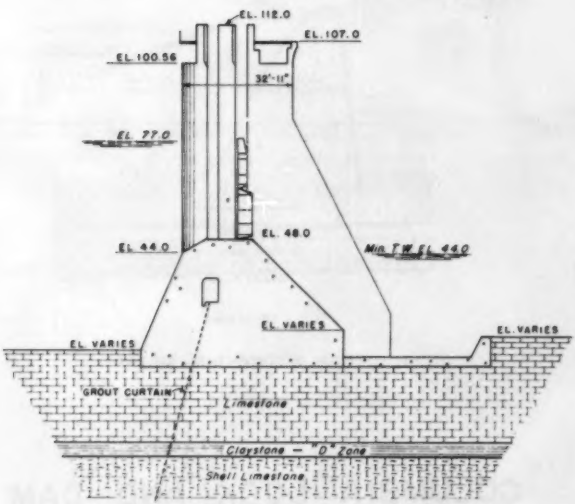
TYPICAL SECTION – THRU SPILLWAY

COLUMBIA LOCK AND DAM

FIGURE 4



PLAN VIEW



TYPICAL SECTION — GATED SPILLWAY

JIM WOODRUFF LOCK AND DAM

FIGURE 5

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

THE DEVELOPMENT OF THE CUMBERLAND RIVER BASIN[a]

Frank P. Gaines, A.M. ASCE,[1] and John T. Dennison,[2] A.M. ASCE
(Proc. Paper 1121)

SYNOPSIS

In the midst of numerous discussions of basin authorities to obtain multiple water uses, the Cumberland River Basin is being developed by the Corps of Engineers, U.S. Army, for multiple water uses. The comprehensive plan includes the development of the Cumberland River and tributaries for navigation, flood control, power development, and other purposes.

INTRODUCTION

Rivers have played an important part in the development of our nation with our first expansion westward being over the inland waterways. Almost 130 years ago, the Congress of the United States realized the value of our water resources and assigned the responsibility for development of our inland and coastal waters to the Corps of Engineers, U.S. Army.

It is interesting to note that the Cumberland River was among the first to receive Federal appropriations for navigation improvement. Following the initial appropriation in 1832, intermittent appropriations were made for open channel work to improve navigable conditions, by the construction of wing dams, dikes, and removal of obstructions. This work, however, was not very effective in providing dependable, year-around navigation.

In 1876 a recommendation was submitted to Congress for the Corps of Engineers to canalize the Cumberland River by the construction of relatively low lift locks and dams. In view of the fact that Nashville was the most

Note: Discussion open until May 1, 1957. Paper 1121 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 5, December, 1956.

[a]Presented at a meeting of the ASCE, Waterways and Harbors Division, Knoxville, Tenn.

1. Chief, Eng. Dir., Nashville Dist., Corps of Engrs., U.S. Dept. of the Army, Nashville, Tenn.
2. Asst. Chief, Eng. Div., Nashville Dist., Corps of Engrs., U.S. Dept. of the Army, Nashville, Tenn.

important port for river traffic, two separate projects were recommended: One for the reach from the mouth of the river to Nashville, Tennessee, a distance of 191 miles, and the other from Nashville to the vicinity of Burnside, Kentucky, a distance of 325 miles. The projects for the river below Nashville were authorized for construction in 1892 and work was immediately begun on the construction of Locks A to F. These locks were completed and placed in operation between 1904 and 1923 (SEE EXHIBIT 1). The project above Nashville was basically adopted in 1886 with details of construction left for subsequent decision. That project provided for construction of locks and dams Nos. 1 to 17 and 21 to secure 6 foot navigation at low water from Nashville to Waitsboro, a point between Carthage and Celina, Tennessee. Locks and dams 1 to 8, together with Lock 21, which gave a harbor to the then thriving river town of Burnside, Kentucky, were completed between 1904 and 1924 (SEE BASIN PLAN EXHIBIT 2 SHOWING ORIGINAL PROJECT IN BLACK). Due to the general reduction in the amount of river traffic, Lock and Dams Nos. 9 to 17 were never started.

The Engineers, under the direction of Congress, continued their study of the river. House Document 38 (the so-called "308 report") of the 73rd Congress was the first to present a really coordinated plan of development in the combined interest of navigation, flood control, hydroelectric power, and related benefits. Although the only construction accomplished in the Cumberland River Basin was the partial rehabilitation of the navigation structures and one local project, I believe that the "308 Report" was the forerunner of the newly accepted conception that a main river and all its tributaries must be dealt with as a complete integrated system and not piece meal. The "308 Report" established the basic plan.

The disastrous flood of 1937 pointedly emphasized that the plan must be acted upon if repetitions of the disaster were to be avoided. As a result, Congress passed the Flood Control Act of 1938 which authorized the improvement and protection of the Ohio River Basin. This plan included six reservoir projects within the Cumberland River Basin. As a result of further directives from Congress, the Nashville District developed the comprehensive plan for the Development of the Cumberland River and Tributaries. This plan was essentially approved by the River and Harbor Act of 1946, the Flood Control Acts of 1936 and 1950, and the River and Harbor Act of 1954.

Basin Description

Formed by the confluence of Poor and Clover Forks, near the City of Harlan in southeastern Kentucky, the Cumberland River is one of the major tributaries of the Ohio River, having a basin area of 17,720 square miles, of which 10,160 square miles are in Tennessee, and 7,560 in Kentucky. Somewhat crescent in shape, the basin has an average width of about 50 miles and an axial length of approximately 350 miles with topography varying from the rugged Cumberland Mountains in the eastern portion to the alluvial valley of the lower Ohio River in the west. The elevations vary from over 4,100 feet above the mean sea level in the mountains to 302 feet in the pool water at the mouth of the river. Flowing in a general southwesterly direction into Tennessee, to Nashville, thence northwestward into Kentucky again the river empties into the Ohio River 58.5 miles above the junction of the Ohio and Mississippi Rivers.

The Comprehensive Plan

The plan adopted by the Department of the Army as providing the best overall development of the water resources of the basin consists of five local protection projects, one flood control only reservoir, and 10 multiple purpose reservoirs. (SEE EXHIBIT 2). In addition to the projects authorized, there are four reservoir sites that warrant future consideration. Each project in the plan will be discussed briefly.

Cumberland, Kentucky, Local Protection Project

The town of Cumberland, Kentucky, is located on the Poor Fork of the Cumberland River approximately 23 miles above its confluence with the Clover Fork. After investigation, the plan considered most feasible for flood relief consists of improving the carrying capacity of the stream by removal of deposits of boulders, gravel, and sand which has accumulated over the years. While this will not prevent damages to the city due to major floods, it will reduce the flood stages. Cumberland undertook and completed this project with local funds.

Middlesboro, Kentucky, Local Protection Project

The Middlesboro, Kentucky, project is divided into two phases, the first of which was authorized in the 1936 Flood Control Act and consists of improvement of a canal and levee system about four miles in length, the purpose of which was to intercept the headwater streams of Bennett's Fork and Stony Fork and divert them away from the business district of the town. This phase of the project cost approximately $820,000 and has provided substantial benefits since its completion in 1940. The second phase of the project consists of a levee along Yellow Creek to be constructed at an estimated cost of $581,000. Upon completion of this levee system, the town will be protected from headwater flooding.

Pineville, Kentucky, Local Protection Project

Pineville, Kentucky, located along the Cumberland River between mile 653 and 655, has an active project made up of a system of levees and floodways along the left bank of the Cumberland for the protection of the main town, and a levee on the right bank for the protection of the Pineville suburb named Wallsend. Federal cost of the project is estimated to be $1,720,000. The levee sections total 4,300 feet in length with average height of 11 feet, and wall section totals 6,000 feet in length with an average height of 8 feet, with two gated structures and three pumping plants. Heights of the levee and wall sections of the protection works are based on the flood of record plus a minimum of three feet of freeboard which will actually provide protection from a flood somewhat greater than the flood of record. Levees and flood walls for local flood protection projects are usually investigated for construction with top profile elevations to afford protection against a standard project flood, which is the maximum reasonably to be expected at some future date. The Pineville project was so investigated and found to be economically feasible; however, the cost of the lands and relocations involved which, by law, must be borne by local interest, was greater than the local people felt that they could finance. Consequently, the top profile elevations were lowered to afford protection against a repetition of the maximum flood of record with three feet of freeboard, as requested by local interest and to be within their financial capabilities. This plan was acceptable to the Corps of Engineers only because an unusual hazard of loss of life is not present and the local interests

are fully aware that they do not have complete protection and must not obtain a false sense of security in their development. Pineville has been subject to frequent and damaging floods. Damages resulting from a flood equal to the 1946 flood (flood of record), under the present stage of development, would amount to over $2,300,000. During the last 37 years, Pineville has been subjected to eleven damaging floods which would have caused damages within the protected works of about $5,700,000 under present conditions of development. Estimated annual costs of the project, including interest and amortization of investment, is $104,400.

Construction of the project was initiated in July 1953, and all of the wall and levee sections, except for a very short wall section at the upper end of the project, is either completed or under contract. It is proposed to award a contract for this short section and the pumping plants in the near future and to complete the project prior to the next flood season.

Barbourville, Kentucky, Local Protection Project

Barbourville is located on the Cumberland River 635.2 miles above its confluence with the Ohio River. The plan of improvement consists primarily of a system of earth levees approximately 18,900 feet in length with an average height of 15 feet, and four pumping plants. Estimated federal cost of the project is $1,780,000 with an estimated annual cost, including interest and amortization of investment, of $98,800. The project is designed to provide protection for the flood of record, plus a 3 foot freeboard.

As in the case of Pineville, the local interests at Barbourville requested that the project be constructed with top elevations to afford protection against a repetition of the maximum flood of record, with freeboard, rather than the maximum flood reasonably to be expected at some future date. Subject to frequent and damaging floods, the city has within the last 39 years suffered 38 such occurrences, including 14 of major proportions which would have caused total losses within the protected area of $5,836,000 under present conditions of development. The February 1956 flood affected over 330 buildings and homes situated within the protected area and resulted in evacuation of 250 families by the National Guard, in addition to those who moved out unassisted.

Construction of the project was initiated June 1955 for the construction of the main levee along the Cumberland River which section will be completed and work initiated on the remaining levee section and pumping plants during the present calendar year.

Unauthorized Projects for Future Consideration

The Laurel project on the Laurel River, the Rockcastle project on the Rockcastle River, and the Devil's Jumps and Helenwood projects on the South Fork of the Cumberland River have not been authorized for construction by Congress. These projects are hydroelectric power projects and would make full use of the water and static head available from these important tributaries of the Cumberland River. Proceeding on instructions of Congress, the Corps of Engineers is presently engaged in a study to determine the economical feasibility of constructing the Laurel, Devil's Jumps, and Helenwood projects. Present estimates indicate a total available capacity of about 85,000 kilowatts at these sites, with approximately 316,000,000 kilowatt hours of electrical energy available annually. Cost estimates are in the process of being developed.

Wolf Creek Reservoir Project

Located on the Cumberland River in Kentucky, the Wolf Creek project is the largest of the headwater developments (SEE EXHIBIT 3) and has been constructed for the primary purposes of flood control and hydroelectric power, being presently in operation for both these purposes. A combined earth fill and concrete gravity dam structure has been constructed with a maximum height of 258 feet. The power installation consists of six generating units of 45,000 kilowatt capacity each giving a rated plant capacity of 270,000 kilowatts. The estimated average annual energy output of this plant is 867,000,000 kilowatt hours. The plant has produced over three billion kilowatt hours of electrical energy since the first unit was placed in commercial production in October 1951. Designated by Congressional action as Lake Cumberland, the reservoir covers 63,530 acres, and has a total storage capacity of 6,089,000 acre-feet. Of this amount, 2,094,000 is reserved exclusively for flood control, and 2,142,000 acre-feet is available for power. The remaining storage comprises the conservation pool. Project cost is $79,178,000.

Construction of the dam and power plant was initiated in August 1941. Work was suspended, however, by order of the War Production Board from August 1943 to September 1946 in the interest of the war effort. With final closure of the dam in December 1950, the project was fully effective for flood control operation. The six power units were placed in operation on the following dates: 6 October 1951; 31 October 1951; 16 December 1951; 17 March 1952; 24 July 1952, and 22 August 1952.

Total annual cost of the project is $4,553,000, including interest, amortization over a 50 year period, operation and maintenance and taxes foregone in the amount of $1,170,000. Estimated Flood Control and Power Benefits are about $6,250,000 annually. Although no credit is taken for recreational use of reservoirs in the test of economic feasibility, these benefits are substantial. For example, the visitations at Lake Cumberland in 1955 were 2,334,736 man-days; there were 410,200 pounds of fish caught, and the total value of business in the area resulting from the project was estimated at $4,459,000, most of which came from fishermen and hunters visiting the reservoir area. Furthermore, these annual benefits are apparently on the increase.

Celina Dam

Since Carthage, Tennessee, is considered the head of navigation on the Cumberland River as far as the foreseeable future is concerned, the Celina Dam, which is to be located on the Cumberland River at river mile 385.8 about 77 miles upstream from the town of Carthage, does not have navigation facilities planned. Power production is the primary purpose of the Celina project, and the plan is for a combined concrete-gravity and earthfill dam about 2,300 feet in length at a maximum height of 86 feet, with three power units having a capacity of 21,600 kilowatts each. The total power plant capacity is 64,800 kilowatts which will produce an estimated annual energy output of 220,000,000 kilowatt hours. The reservoir will extend 75.2 miles upstream to the Wolf Creek Dam. Estimated to cost $34,300,000, the project is planned to produce hydroelectric power by use of the flows regulated by the Wolf Creek project and by use of the fall of the river between the adjacent upstream and downstream projects (Wolf Creek and Carthage). Definite design studies have been completed; however, preparation of detail plans and

specifications is dependent upon the desires of Congress, as expressed through appropriation acts.

Dale Hollow Reservoir Project

Constructed for the primary purposes of flood control and hydroelectric power, the Dale Hollow project is in operation for both purposes. (SEE EXHIBIT 4). The dam is located on the Obey River about 7.3 miles above its confluence with the Cumberland River, is a concrete-gravity structure 1,717 feet in length, and has a maximum height of 200 feet. Power facilities consist of three generating units of 18,000 kilowatt capacity each, giving a total plant capacity of 54,000 kilowatts. Estimated average annual energy output of the plant is 127,000,000 kilowatt hours, and since initial commercial power production in December 1948, over 880,000,000 kilowatt hours of electrical energy have been produced. The reservoir covers an area of approximately 30,990 acres and has a total storage capacity of 1,706,000 acre-feet of which 353,000 acre-feet are reserved exclusively for flood control and 496,000 acre-feet for power production.

Construction of the dam and power plant was initiated in March 1942, but curtailed in January 1943 by order of the War Production Board. The dam structure was completed for flood control under the curtailment order in October 1943, although construction of the power plant was not resumed until July 1946. First power was effected when the first generating unit was placed in operation in December 1948, with the second unit being placed in operation in January 1949, which completed the initial installation. Installation of the third and final unit was accomplished during the period March 1952 to November 1953. Project cost is $25,989,000.

Carthage Dam

Carthage Dam, which is to be located on the Cumberland River at mile 313.5, approximately 5 miles upstream from Carthage, is designed to produce hydroelectric power with the structure layout planned so that future navigation facilities can be added at some future time if economically justified. The dam will be a concrete gravity and rolled earth fill structure approximately 1,280 feet in length with a maximum height of 98 feet, behind which the reservoir will extend 72.2 miles upstream to the site of the Celina dam.

A power installation of four units with a 23,000 kilowatt capacity each is planned. The 92,000 kilowatt power plant will produce an annual average of 310,000,000 kilowatt hours of electrical energy. Estimated cost of the project is $40,000,000.

Definite design studies have been completed, but development of detail plans and specifications depends upon Congressional appropriations.

Center Hill Reservoir Project

The Center Hill project costing approximately $44,345,000 (SEE EXHIBIT 5) is on the Caney Fork River with the dam located about 26.6 miles from the confluence of the Caney Fork with the Cumberland River. Constructed for the primary purposes of flood control and hydroelectric power, the project is in operation for both purposes. The main dam is a combination concrete-gravity and earthfill structure 2,160 feet in length, with a maximum height of 250 feet, while the saddle dam, constructed across a low profile in the reservoir rim, is a rolled-earth dike or dam, 770 feet in length on the right bank near the dam site. Covering an area of approximately 23,060 acres, the

reservoir has a total storage capacity of 2,092,000 acre-feet, of which 762,000 acre-feet are reserved for flood control and 492,000 acre-feet for power. The power installation consists of three generating units of 45,000 kilowatt capacity each, giving a total plant capacity of 135,000 kilowatts and an estimated average annual energy output of 351,000,000 kilowatt hours. Since initial commercial power production in December 1950, over 1,700,000,000 kilowatt hours of energy have been produced.

Construction of the Dam and power plant was initiated in March 1942 with work being suspended by order of the War Production Board from March 1943 to January 1946 in interest of the war effort. With final closure of the dam in November 1948, the project was fully effective for flood control operation. The power plant was completed in November 1951 with the three power units being placed in operation respectively on 11 December 1950, 17 January 1951, and 11 April 1951.

Old Hickory Project

The Old Hickory project is under construction on the Cumberland River with the dam, lock, and power plant located at mile 216.2, approximately 3 miles downstream from the town of Old Hickory and 23 miles upstream from Nashville, Tennessee, which extends the reservoir 97.3 miles upstream to the site of the Carthage dam. The project is designed to produce hydroelectric power from flows regulated by existing upstream reservoirs and by use of the fall of the river between adjacent upstream and downstream projects (Carthage and Cheatham) and to provide for continuance of the nine foot navigation channel on the Cumberland River upstream to the town of Carthage, Tennessee. (SEE EXHIBIT 6). Structures consist of an earth dam section, an 84 x 400 foot navigation lock, concrete spillway section in the main river and a conventional indoor type powerhouse. Four 25,000 kilowatt power units will give the project a total installed capacity of 100,000 kilowatts with an estimated annual output of 420,000,000 kilowatt hours. Total combined length of the structures is 3,750 feet normal to the river flow; maximum height of the structure is 98 feet and the normal lock lift is 60 feet. Estimated cost of the project is $48,800,000.

Construction was initiated in January 1952 and since that time the lock and spillway sections have been completed except for the permanent lock sill which will be placed as the reservoir pool is raised. The power plant is under construction with the first power unit scheduled for production in December of this year. Unusually heavy usage of the reservoir is expected due to the proximity to Nashville, Tennessee, which will result in a very large recreational value, although the recreational value was not used in the economical feasibilities studies by the Corps of Engineers.

Locks and Dams 3 and 4 of the old and existing navigation project above Nashville have already been inundated by the Old Hickory pool which is presently being operated about 16 feet above the spillway crest. Locks and Dams 5, 6, and 7 will be demolished or completely inundated when the Old Hickory pool is raised to the final operating levels.

Stewarts Ferry Project

Plans for Stewarts Ferry project locate the dam and power plant on the Stones River 6.8 miles above its confluence with the Cumberland River only a few miles above Nashville, Tennessee. Estimated to cost approximately $23,400,000, the project is designed to provide flood control and hydroelectric power and together with the completed Wolf Creek, Dale Hollow, and

Center Hill projects, will reduce all floods of record to a no damage stage at Nashville. The reservoir will have a total capacity of 723,000 acre-feet of which 421,000 is reserved for flood control during the flood season and 331,000 acre-feet during non-flood season, 194,000 reserved for a conservation pool, and the balance available for power production. A single 13,500 kilowatt unit will be installed which will produce about 52,000,000 kilowatt hours annually. The project is in the planning stage, but it is expected that the dam will be a combination rolled earthfill and concrete-gravity structure having a total length of 1,875 feet, with a maximum height of 146 feet.

Three Islands Project

The proposed Three Islands dam now in the planning stage will be located on the Harpeth River, 6.3 miles above its confluence with the Cumberland River, and is being planned for the primary purposes of Flood Control and Power at an estimated cost of $21,900,000. A combination rolled earth fill and concrete gravity dam having a length of 2,600 feet and a maximum height of 142 feet will be combined with a power plant containing ultimate power installation of 18,000 kilowatts (two units rated at 9,000 kilowatts each) which will produce an average of 54,000,000 kilowatt hours annually. The reservoir will contain 715,000 acre-feet of water, 351,000 acre-feet of which will be reserved for flood control storage, 152,000 for power production and 212,000 acre-feet for conservation.

Cheatham Project

Now under construction on the Cumberland River is the Cheatham project (SEE EXHIBIT 7) with the dam located at mile 148.7, 9 miles below Ashland City, Tennessee, and 42 miles downstream from Nashville, Tennessee. When completed, the pool will extend 67.5 miles upstream to the Old Hickory dam, eliminating Locks A and 1, which have already been demolished, and Lock 2 which will be demolished this year. Estimated cost of the project is $31,700,000 which includes facilities for production of hydroelectric power from flows regulated by upstream reservoirs, and by use of the fall in the river between the adjacent projects (Old Hickory and Lower Cumberland) and facilities to provide navigation through Nashville, Tennessee, to the Old Hickory Lock. The structures have a maximum height of 75 feet and consist of a 110 x 800 feet modern lock with a normal lift of 26 feet, a crest gage controlled concrete spillway and a conventional type powerhouse. Power installation of three 12,000 kilowatt units will provide for a total plant capacity of 36,000 kilowatts and produce an average of 160,000,000 kilowatt hours annually.

Construction of the project was initiated in April 1950. The lock and dam have been completed and were placed in permanent operation during August 1954 at which time the pool was raised to elevation 380 (the permanent operating pool will be between 382 - 385).

Final construction contract for construction of the power plant has been awarded and first power production is scheduled for October 1957.

Rossview Project

The only reservoir project in the system being planned for flood control purposes only is the Rossview project, located on Red River 17.4 miles above its confluence with the Cumberland River. As planned, the structure consists of a rolled earth fill dam in the main stream, a saddle type uncontrolled spillway and concrete-lined tunnels for diversion and sluicing. The reservoir

will have a total storage of 372,000 acre-feet, of which 352,000 will be used exclusively for flood control.

Barkley Dam (Lower Cumberland Project)

The $167,000,000 Barkley Dam (Lower Cumberland) project now in the design stage is to be constructed at mile 30.5 on the Cumberland River as a key project in the development of the basin. This project will replace five of the obsolete locks now in operation (Locks B to F, inclusive) and will consist of a dam approximately 7,785 feet in length, exclusive of the lock, with a maximum height of 130 feet; a lock 110 feet wide and 800 feet long with a normal lift of 57 feet; and a power plant with a capacity of 130,000 kilowatts made up of five units rated at 26,000 kilowatts each. Power production will be an average of 600,000,000 kilowatt hours of electrical energy annually. Included in the plans is a provision for the construction of a canal to connect Lake Barkley (Lower Cumberland reservoir) with the pool of the Kentucky Dam, located on the Tennessee River, for the purpose of navigation and diversion of flow from each reservoir as required for flood control or power production. The pool formed by the Barkley (Lower Cumberland) dam will extend upriver to the Cheatham Dam.

It is interesting to note that where it takes an average river tow about 20 hours to lock through the present obsolete locks being replaced by the Barkley (Lower Cumberland) lock, the same tow will be able to lock through the Barkley Lock in a matter of minutes.

SUMMARY

The comprehensive development of the Cumberland River, as outlined, will involve an estimated total Federal expenditure of $534,000,000 with an annual cost of $28,500,000 for maintaining the system, including interest on investment, amortization of investment and taxes foregone on the power features. This figure is based on the amortization of investments in 50 years although it is believed that the structures will last much longer. For these expenditures average annual benefits for flood control will be in the order of $6,650,000. While substantial flood control benefits will be realized in the Cumberland River Basin (e.g., the system will reduce the flood of record to a no damage stage at Nashville, Tennessee), the greatest value of the system for flood control accrues from the holdouts which provide sizable flow reductions on the lower Ohio and Mississippi Rivers. By discharging Cumberland River flows in advance of the flood crests on the Ohio and Mississippi and then storing the Cumberland River flows during the flood crests, maximum utilization of the Cumberland River projects will be obtained. The navigation benefits will be $4,990,000 and the power values will approximate $25,100,000 annually, giving an annual benefit to the system of $33,800,000. This then means that for every one dollar spent, about $1.18 in benefits will be obtained. These benefit figures are tangibles of the kind that appeal to business men, bankers and the cold eye of Government Comptrollers, and others who handle Federal funds.

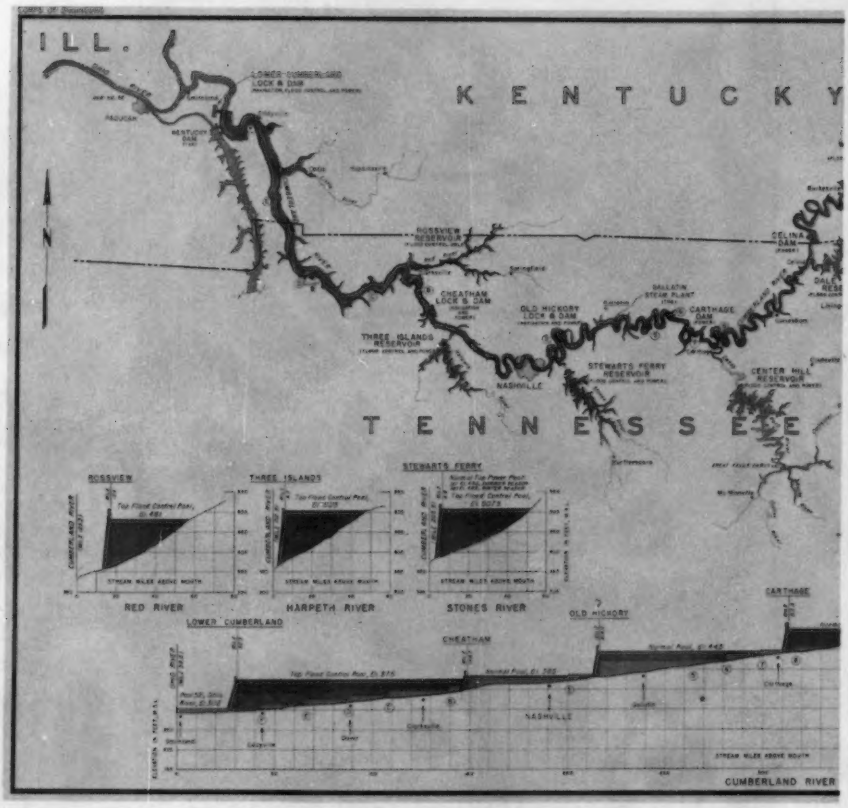
Supplementary to these hard-money figures are intangibles which are hard to determine in dollars and cents, but which have a profound effect on the happiness and well-being of a great number of people. Such include improved conservation of our water resources, the development of additional fish and wildlife areas, pollution abatement, improved water supply and extensive

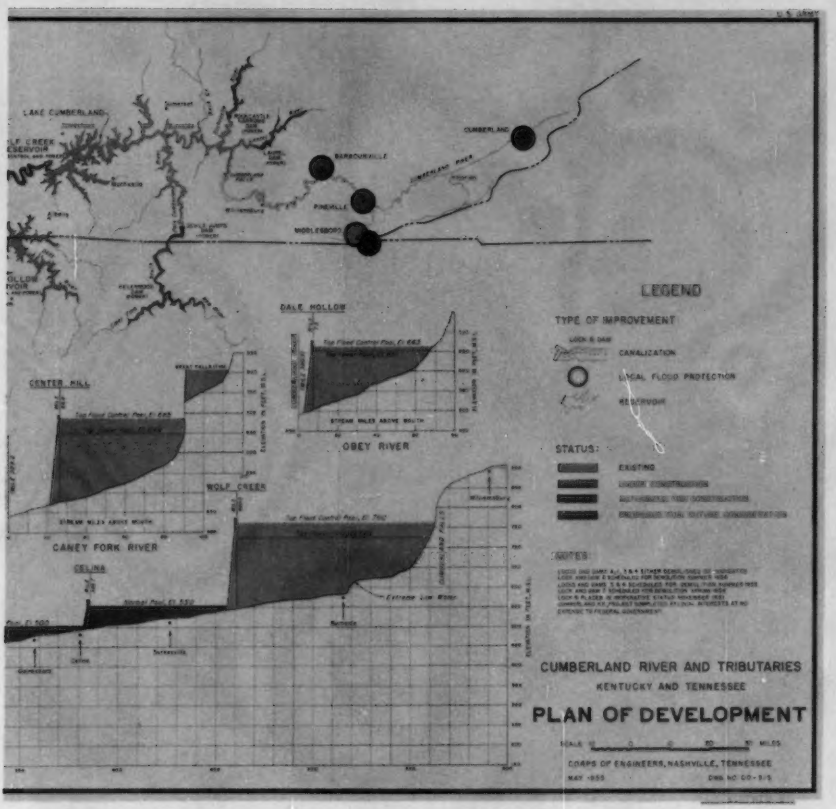
recreational benefits. The low water flows resulting from the operation of the headwater projects are in the order of 3 or 4 times greater than the minimum natural flows. This provides a better water supply for all uses and reduces stream pollution during the summer (e.g., due to this increased flow, the sewage treatment plant planned for construction by the City of Nashville, Tennessee, will provide for primary treatment only, thus saving approximately $250,000 on secondary treatment). The increased flows will provide benefits all the way to the Gulf of Mexico. Analysis made by others has indicated that the recreational and related benefits alone would more than justify construction of all the reservoir projects in the basin, including the power features.

7 Exhibits

1. Construction Lock No. 1, Cumberland River
2. Plan of Development, Cumberland River and Tributaries
3. Wolf Creek Dam, Cumberland River
4. Dale Hollow Dam, Obey River
5. Center Hill Dam, Caney Fork River
6. Old Hickory Lock and Dam, Cumberland River
7. Cheatham Lock and Dam, Cumberland River

#595-Lock 1, River wall and cofferdam looking downstream

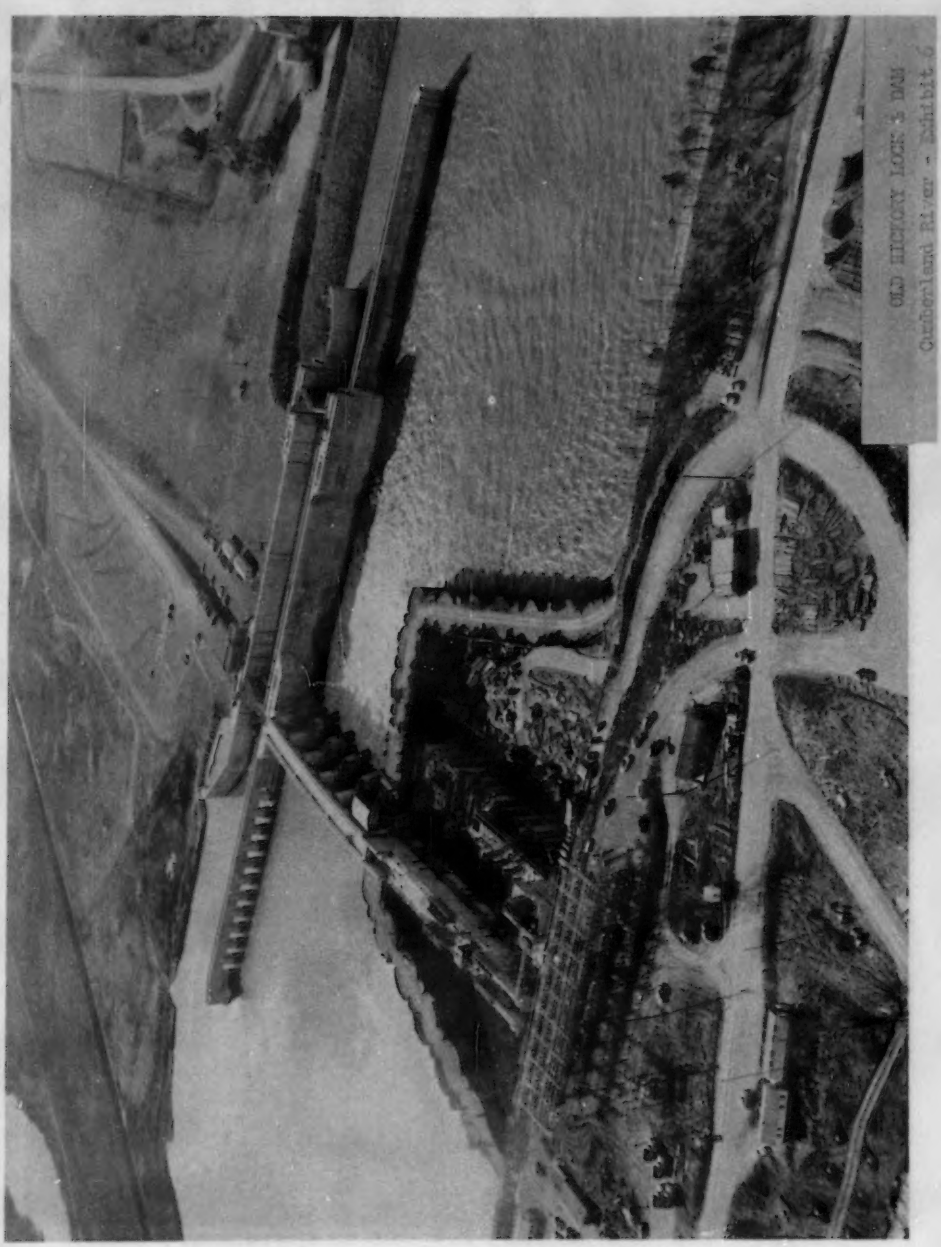

ILL.
KENTUCKY
TENNESSEE
LOWER CUMBERLAND LOCK & DAM
KENTUCKY DAM
PADUCAH
BARKLEY DAM
ROSSVIEW RESERVOIR
CHEATHAM LOCK & DAM
OLD HICKORY LOCK & DAM
DALLATIN STEAM PLANT
CARTHAGE DAM
CELINA DAM
THREE ISLANDS RESERVOIR
NASHVILLE
STEWARTS FERRY RESERVOIR
CENTER HILL RESERVOIR
ROSSVIEW
THREE ISLANDS
STEWARTS FERRY
RED RIVER
HARPETH RIVER
STONES RIVER
LOWER CUMBERLAND
CHEATHAM
OLD HICKORY
CARTHAGE
NASHVILLE
CUMBERLAND RIVER

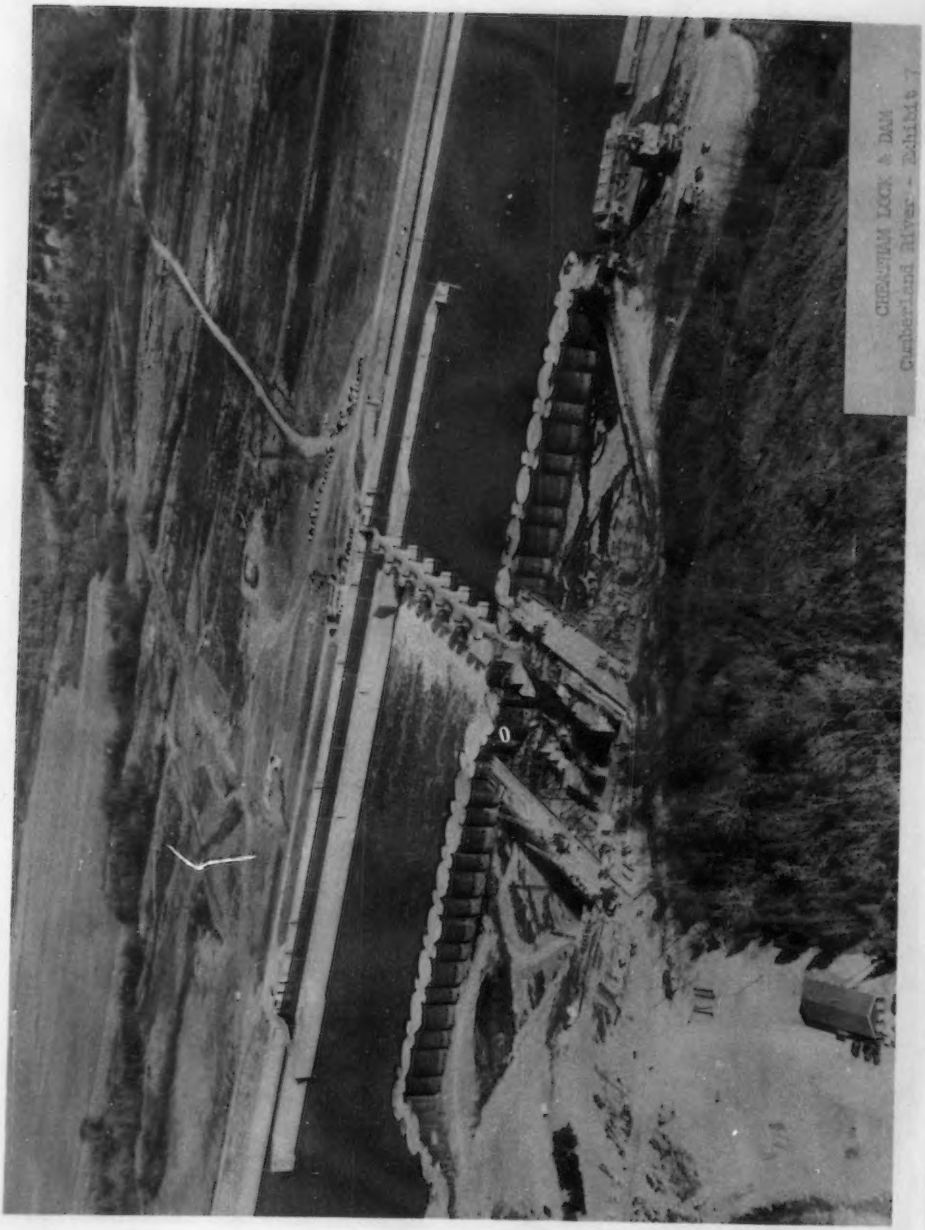

LAKE CUMBERLAND
BURKESVILLE
CUMBERLAND
PINEVILLE
WILLIAMSBURG
DALE HOLLOW
OBEY RIVER
CENTER HILL
CANEY FORK RIVER
WOLF CREEK
CELINA
LEGEND
TYPE OF IMPROVEMENT
CANALIZATION
LOCAL FLOOD PROTECTION
RESERVOIR
STATUS:
EXISTING
CUMBERLAND RIVER AND TRIBUTARIES
KENTUCKY AND TENNESSEE
PLAN OF DEVELOPMENT
CORPS OF ENGINEERS, NASHVILLE, TENNESSEE

WOLF CREEK DAM
Cumberland River - Exhibit 3

DALE HOLLOW DAM
Obey River - Exhibit A

CENTER HILL DAM
Caney Fork River - Tenn.


OLD HICKORY LOCK & DAM
Cumberland River - Exhibit 6


CHEATHAM LOCK & DAM
Cumberland River - Exhibit 7

Paper 1122 WW 5

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

MODERN TOWBOAT AND BARGE DESIGN

Harry B. Dyer,[1] M. ASCE
(Proc. Paper 1122)

As an introduction to the discussion of modern towboat and barge design, it seems appropriate to go back into a little of the history of our river transportation. Prior to 1811, all traffic on the inland rivers was by powerless craft. The first successful power boat to operate on the inland rivers was the steamboat NEW ORLEANS, built in Pittsburgh in 1811. There was an immediate opportunity for the rapid growth of steamboat navigation in the development of the recently acquired Louisiana Purchase, but this growth was obstructed for a number of years by the attempts of various steamboat builders to create a monopoly of navigation on the western rivers. Edward Livingston, acting for the Livingston-Fulton Company, approached the legislatures of the western states, seeking franchises for the exclusive development of steam navigation, and he was successful in obtaining such a franchise in Louisiana.

The first flat-bottomed steamboat, which served as a model for most of the later Mississippi River boats, was designed by Henry Shreve. It was in such a steamboat, called the WESTERN ENGINEER, that Major Long made the first steamboat trip up the Missouri River, traveling from Pittsburgh to the Rockies in the year 1819. Descriptions say that, in order to mystify the Indians along the Missouri, the bow of the boat was shaped to imitate the head and neck of a serpent, and the smoke issued from a stack concealed in the serpent's head.

The Northwest Ordinance of 1787 first recognized the necessity for maintaining free and unrestricted navigation on the Missouri River and its tributaries. Many subsequent resolutions and decrees bore out this inherent freedom of all the people to the unrestricted use of our inland waterways; and, when some states attempted to issue monopolistic franchises to certain operating companies, Henry Shreve, recognizing the necessity for unrestricted navigation, challenged the various franchise monopolies and carried the matter before the Supreme Court of the United States. In 1824, Chief Justice John Marshall rendered a memorable decision, proclaiming the "navigable waters of the United States forever free for the use of the people as highways of commerce."

Note: Discussion open until May 1, 1957. Paper 1122 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 5, December, 1956.

1. Pres., Nashville Bridge Co., Nashville, Tenn.

Read in the light of the historical background, it is clear that freedom of navigation guaranteed by the Northwest Ordinance and other basic documents was intended to insure untrammeled use of the Western Rivers to all our citizens. The underlying policy was not merely to prevent the various states from imposing discriminatory restraints but to deny to any governmental body, foreign or domestic, the power to obstruct, limit or tax the right of all the people to navigation on the rivers as common highways. This view is summed up as follows in President John Tyler's Veto Message of June 11, 1844:

> "It (the Mississippi River) belongs to no particular State or States, but of common right, by express reservation, to all the States. It is reserved as a great common highway for the commerce of the whole country. To have conceded to Louisiana, or to any other State admitted as a new State into the Union, the exclusive jurisdiction, and consequently the right to make improvements and to levy tolls on the segments of the river embraced within its territorial limits, would have been to have disappointed the chief object in the purchase of Louisiana, which was to secure the free use of the Mississippi to all of the people of the United States. Whether levies on commerce were made by a foreign or domestic government would have been equally burdensome and objectionable. The United States, therefore, is charged with its improvement for the benefit of all, and the appropriation of governmental means to its improvement becomes indispensably necessary for the good of all."

During this period, steam navigation increased by leaps and bounds. In 1829, the War Department built the first snag boat. From this beginning, the Corps of Engineers for the United States Army shouldered and carried the burden of keeping the channel clear.

It was in this same year 1829 when steam navigation was definitely established on the Missouri River, and it marked the beginning of the expansion period in which the Missouri River became the principal travelway for early access to the West.

Early settlements had been located on the rivers. Now, cities grew up on them. Industry revolved about the ports. In the 1830's, carrying passengers became the main source of revenue in river transportation. It was the day of steamboat races and high speed records. Some outstanding records were made before 1860, and the famous contest between the ROBERT E. LEE and the NATCHEZ in 1870 from New Orleans to St. Louis is the most celebrated of all speed duels between boats. The boats left the wharf at New Orleans at 4:55 p.m. on June 30, 1870; and the ROBERT E. LEE reached the wharf at St. Louis on July 4, 1870, at 11:25 a.m., 6 hours and 33 minutes ahead of the NATCHEZ, which had been detained in a fog downstream for 6 hours. That record of 3 days, 18 hours, and 30 minutes had not been approached until March 1949, when the diesel-powered HARRY TRUMAN with one barge in tow made the same run in 3 days and 20 hours.

The packet boat industry on the Mississippi and Ohio Rivers and their tributaries was one of the foremost business enterprises of the country. Commerce and industry were largely dependent on water transportation for the movement of raw materials and finished products, as well as passengers. The only other transportation available at that time being a few scattered railroads and animal-drawn vehicles. The steam packet boats of that day moved most of the cotton from the producing Southern States, the timber and

lumber from the forests and saw mills, and much of the coal from the mines to the consuming areas.

But, even while the packet boats were in their heyday, there were river men who realized that the most effective means of using the river was in the transportation of bulky cargo in barges pushed in fleets by a towboat. By 1870, fleets of barges became a common sight; and, because oil and coal were found convenient to the rivers, these commodities then as now furnished a great proportion of the tonnage. In the early part of this century, great fleets of wooden coal barges were assembled in the Upper Ohio Valley and towed to New Orleans. The steamer SPRAGUE in February, 1907, took a tow from Louisville to New Orleans composed of 62 wooden coal barges, grossing nearly 70,000 tons, about 7 acres of barges.

But, with the development of the railroads toward the end of the 19th century, even this traffic began to decline rapidly. The railroads furnished faster service and had greater flexibility for serving those communities not blessed with navigable rivers and waters. As they grew, with much assistance from the Government in the way of land grants and financing, the railroads successfully undertook freight rate wars with the packet boats and, as was to be expected, won most of them. The results were obvious; the packet boats were forced out of business over the period beginning about 1890 and ending two decades later.

Thus, by 1910, the railroads, to all practical purposes, were enjoying a monopoly on freight and passenger transportation in this country. A few of the larger industrial and mining companies and corporations continued to maintain and operate private fleets of towboats and barges on the inland waters of the nation, but it was practically impossible for the general public to buy for hire water transportation during the first 20 years of the present century. The packet boats could not exist in the face of such competition from the railroads, and their owners did the only thing they could do—they quit.

The First World War struck this country in 1917; and, by the following year, the railroads, then under Government management, had failed to move properly the heavy traffic necessary for the prosecution of the war. The neglected waterways were unable to supplement the rail system and a demand arose for the rehabilitation of water transportation in the Mississippi Valley.

Hastily, under the direction of the Railroad Administration, a Federal Barge Line was formed, and boats and barges were acquired wherever available, including some chartered from the Army Engineers. With this small fleet, operations began in September 1918. Soon the demand for this service resulted in the construction of six large twin-screw steam towboats and 40 large, 2,000-ton steel barges.

Thus began the revival of water transportation on the inland rivers and waters of the nation. Private initiative and capital followed the Government into the barge line business as we know it today, and the growth of that business and industry has been phenomenal during the past 25 years. The old wooden barges began to disappear and were replaced by steel barges. The wooden hull towboats were likewise replaced by steel towboats. Most of the towboats were of the sternwheel type; but, after the fleet of screw-propelled Federal Barge Line boats proved that the propeller could be satisfactorily applied to river type towboats, more of these began to appear as replacements for the paddlewheel boats. This change-over to propeller-driven towboats was hastened by the advent of the diesel engine, because the

transmission of any appreciable amount of power from the diesel engine to the paddlewheel involved many mechanical complications, which never were satisfactorily solved. The diesel engine was, however, ideally suited to driving the propeller shaft, and this has now become the accepted type of towboat drive.

Between 1925 and 1945, there was a marked increase in the amount of river traffic and tonnage carried on the rivers, but there were no unusual changes in the general design of either towboats or barges. The average barge became larger; but, for the most part, the design was about the same. It was during this period that the so-called jumbo size of barge came into general use. The size of the jumbo barge is 195' x 35', and it was called jumbo because it was so much larger than the so-called standard hopper barges, which were 175' x 26'. Both of these barge sizes were based on fitting the barges into the 600' x 110' locks on the Ohio River. Four of the standard barges will go in the locks abreast and three of the jumbo barges will go in the locks abreast, with about the same amount of clearance in each case. As the tows became larger, it soon became apparent that the jumbo barges were more economical, because only 3 lengths of the standard barges could be put into the locks, while 3 lengths of the jumbo barges could also be put in the locks. A single locking, therefore, could accommodate 12 standard barges or 9 jumbo barges; but, since the 9 jumbo barges could carry 12% more cargo than the 12 standard barges, it was apparent that the larger barges had many advantages.

As more and more oil began to be transported by barge, the jumbo size of barge was adopted as the most common size of oil barge, and soon the 195' x 35' oil barge came to be known as a standard oil barge. Most of the oil moved upstream from Louisiana, so the size of a towboat had to be governed by the number of barges which could satisfactorily be pushed upstream in the heavy current of the Mississippi River. The average towboat during this period ranged from 1200-HP to 1600-HP, and such a boat could take six of the standard 195' x 35' oil brages up the Mississippi River, usually averaging about 2-1/2 miles per hour. This upstream speed of 2-1/2 MPH in the Mississippi River was accepted as about all that could normally be expected or justified. The current in the Mississippi averaged 4 to 5 MPH which meant that, in order to attain the 2-1/2 MPH upstream, it was necessary to actually push the tow about 7 MPH. Attempts were made to increase the pool towing speed about 7 MPH, but it required such a tremendous increase in horsepower that it was not found to be economical. Another reason for not increasing the pool towing speed above about 7 MPH was that the standard barges, if heavily loaded, would dive if they were pushed any faster.

Many people cannot understand how a barge can dive, but it happens quite often with old style barges. When a barge with an ordinary rake and not much sheer is loaded and pushed through the water, it causes the water level ahead of the barge to rise. As the speed of the barge is increased, the water level ahead rises higher and higher until the water finally comes over the forward end of the barge. When this occurs, the weight of the water on the forward end of the barge pushes the barge down more, and the water piles up higher, and very often the barge goes down and rams the river bottom.

Private towboat operators had usually grown up in the business and had learned to accept these limitations on towing speed; and, so, no real progressive changes were made.

At the end of the war in 1945, the Nashville Bridge Company, which had

been a builder of boats and barges since 1915, suggested to some of its customers that it might be possible to design a type of barge fleet which could be economically towed at a faster speed. No towboat operator had been successful in such an attempt, and none of them seemed to want to spend their money on a pure experiment; but, finally, the Ingram Products Company, of Nashville and St. Paul, agreed to try it and placed an order for a two-barge, streamlined, integrated unit, which was the first of its size and type to go into service on the Mississippi River System. Although the barges were pushed by an obsolete, sternwheel steamboat, the results were surprising. It was immediately apparent that, if the towboat design could be revised with particular emphasis on fine hull lines for high speed, then the results that could be obtained would be truly remarkable. The same builder immediately set to work to construct such a boat and another pair of barges; and, early in 1946, the 1600-HP diesel towboat CODRINGTON and two integrated, streamlined, model-end barges were delivered to Canal Barge Company of New Orleans and began to make records that had never been heard of. The 2-1/2 MPH average upstream towing speed in the Mississippi River jumped to more than 5 MPH, which was more than a 100% increase. It was the beginning of the present era of high-speed, integrated barge towing on the Mississippi River System. The CODRINGTON passed every towboat on the River; and, as the news spread, the other operators took notes; and soon orders for similar equipment were being placed by operators from Houston to Pittsburgh and Minneapolis.

During the next few years, the need for increasing the capacity of these integrated units became apparent, and the individual barge length was increased to as much as 290'. The barge width was set at not over 50', since this was the maximum width which could operate through the 52' locks in such rivers as the Cumberland, the Ouachita, and the Warrior-Tombigbee. Two of these large barges would carry about 6,000 tons—or 45,000 barrels—of oil. This was about the maximum practical tow for boats of the CODRINGTON class. It soon became apparent that it would be practical to go beyond the tonnage and speed of this 2-barge unit. It was then that what is known as the box barge was developed as a completely rectangular barge section to fit in between the two streamlined end-section barges and increase the overall length and capacity without materially increasing the towing resistance. By the addition of two or more box barges, the capacity of a single width, integrated fleet could be increased to 10,000 or 12,000 tons; and this has now become common in the oil-towing business. The horsepower of towboats operating with single width, integrated fleets of this type varies up to 3200-HP and pool towing speeds sometimes exceed 12 MPH.

One of the problems encountered in operating such long fleets was that of controlling the forward end of the fleet from the towboat far astern, particularly when entering locks or negotiating bridges, especially where cross currents occur. This problem has been solved by the application of what is known as bow steering. Bow steering has been successfully applied to the lead barge of an integrated fleet by mounting a retractable propeller transversely in the bow of the lead barge and driving this propeller with a diesel engine of about 200-HP. The complete control is actuated electrically from the pilot house of the towboat so that, with a tow as long as 1200-ft., the pilot in the towboat at the aft end of the fleet can gently guide the forward end into a narrow lock even when there are bad cross currents at the lock entrance.

Although it was felt by many operators that the ultimate limit of efficiency

had been reached in this long, single-width, integrated fleet, including a towboat of 3200-HP, it was again a Nashville firm which took the lead in applying this principle to new and larger fields. Heretofore, the use of large, integrated barges had been applied solely in the oil-towing trade; and it was Nashville Coal Company which first placed in service completely integrated coal barges 290' x 50'. But, they were not content with the capacity of a single-width fleet, and so they doubled the width into an integrated fleet of 22,500 tons capacity. Their first two towboats had only 3200-HP each; and, although these boats can satisfactorily handle this much tonnage in pool water, they cannot make adequate speed upstream in the lower Mississippi River. Consequently, they have now under construction at Nashville a new towboat of 5400-HP; and this will be the most powerful river towboat on the inland waterways of this country. It will normally handle nine of the 290' x 50' barges, or fifteen integrated jumbo barges with a capacity of 22,500 tons in either fleet. The speed of operation will be comparable to the speed of the single width, integrated fleets hereinbefore described.

Although most of the boats presently being constructed are twin-screw boats, a number of triple-screw boats have recently appeared; and one large boat for use on the Missouri River has four propellers and four engines. None of these triple-screw or four-screw boats has as much total power as the 5400-HP, twin-screw boat previously described. The use of three and four propellers was for the purpose of reducing the draft of the boat for operation in shallow water.

Progress has been made not only in increased tonnage and increased speed through the water, but in increased speed of making up tow, loading and unloading cargo and in operating through thick weather and fog. All of these factors have tended to reduce the cost per ton-mile for towing in spite of the astounding increase in the cost of equipment, labor, and supplies; and, because of this fact, the volume of tonnage moved on the rivers has increased tremendously. As an illustration, the volume of tonnage handled by barges on the Inland Waterways System in 1920 was about 50 million tons. In 1935, it had risen to over 77 million tons; in 1945, to 123 million tons; and, in 1953, to over 170 million tons.

Some of the improvements on towboats which have contributed to the recent increase in efficiency are listed as follows:

1) Much finer hull lines with smoother hull surface and eliminating all possible source of skin friction or turbulence, particularly in the line of flow to and from the propellers.

2) The use of nozzles around the propellers. The full nozzle, known as the Kort nozzle, is advantageous with heavier tows whose speed through the water is in the low or medium speed range. It is particularly well suited to the large tows of the common carrier fleets where the fleet is made up of many different sizes and types of barges which cannot be integrated. The purpose or function of the nozzle is to collect and confine the water around the propeller and to direct the water flow aft from the propeller and thus prevent throwing the water out centrifugally from the propeller in cases where high slip at low speed is encountered. The top half of the nozzle is much more effective in this respect than the bottom half because of the fact that, on tunnel-type river towboats, the water pressure at the top of the propeller is often negative, due to the tunnel which permits part of the propeller to be above the normal water line.

The nozzle collects and directs the water into the propeller and helps to maintain a positive pressure at the propeller. The lower half of the propeller is always submerged sufficiently so that there is no problem of maintaining a positive water pressure; and, therefore, the lower half of the nozzle has practically no benefits in this respect. On high-speed, streamlined, integrated tows, the propeller slip is much less than on heavy slow tows; and the advantage of the nozzle diminishes as the speed increases. A point is reached where the bottom half of the nozzle, due to its resistance through the water at high speed, becomes a hindrance rather than an asset; and it was this fact that prompted the development by Nashville Bridge Company of what is termed the semi-nozzle, or NABRICO Nozzle. This nozzle comprises the top half only of a Kort nozzle. The bottom half is completely omitted so that all the advantages of the top half are retained without the disadvantages of the bottom half. Other advantages of this NABRICO Nozzle in addition to higher speed are better maneuverability of the boat and less damage from drift which can become confined in the full nozzle with no chance of being thrown out at the bottom.

3) Contra Guide rudders, which help reduce rudder drag by being shaped to conform to the flow lines of the water leaving the propellers.

4) Smoother, completely machined propellers with airfoil section and heavier tips, making them more efficient and less subject to damage caused by hitting drift or gravel.

5) Split strut hubs with the entire stern tube and tailshaft assembly, including bearings and gland, removable as a unit. By having a spare assembly on hand, a complete change can be made without the necessity of holding the boat on the drydock while the shaft is re-sleeved and new bushings are fitted and bored.

6) Skin cooling for engine cooling water which completely eliminates the use of any muddy, sandy, or contaminated river water and also eliminates the trouble caused by clogging of sea chests with ice in winter. The interval type of skin cooler known as the NABRICO type has the advantage of leaving the external shell of the boat clean and free from any added resistance, and it also adds strength and safety to the hull by reason of its double-bottom effect. In addition, it is much less subject to damage from grounding or collision than any other type of external skin cooler.

7) Improved diesel engines with supercharging which operate satisfactorily at much higher engine speeds and higher brake mean effective pressures. This permits much more power to be installed within the same space and weight limitations.

8) Automatically regulated engine water temperature which eliminates many troubles caused by improper or changing temperatures.

9) Improved engine lubricants with additives to prevent sludging.

10) Improved oil filtration with automatic temperature control.

11) Simplified pilot house control of the engines, which operates with the same ease and simplicity as an ordinary ship's telegraph.

12) Public address systems, which enable the master to be heard in any section of the vessel or out on the head of the tow or on shore.

13) Radiotelephone equipment, which provides communications from vessel to shore or with any other vessel.

14) Radar equipment, which not only is an aid in ordinary night operations, but permits operation in heavy rainstorm or fog.

15) Gyroscopically controlled automatic steering, which is of greatest

assistance in maintaining a straight course when operating by radar under conditions of restricted visibility.

16) Retractable pilot houses have been developed which can be quickly lowered for passing under the bridges in the Chicago area.

17) Special, heavy-duty, air-powered, wire rope winches are replacing steamboat ratchets for making the towboat up to the barges.

Recent barge improvements include the following:

1) The streamlined model ends have become longer and more sheer has been added. The latest model-end barges have 90' of model for a 50' barge width and as much as 3'-3" of sheer. They will not dive at any speed yet attained.

2) Special, spring-protected, retractable interlocking pins have been developed for maintaining the integrated barges in exact position.

3) Special, quick-connecting winches and wire leads have been developed for getting the barges quickly into exact position and holding them sufficiently for the tow to get underway while additional connecting wires are added later.

4) Improved pumping units enable a 24,000-bbl. barge to be pumped out in 6 hours; and, where adequate shore lines and facilities are available, four such barges can be pumped out simultaneously.

5) Special derricks on the barges facilitate handling the heavy hoses to speed up the loading and unloading processes.

6) Brighter and more dependable running lights fed by current from aboard the towboat make night navigation safer.

7) Special provisions for carrying large quantities of fuel oil aboard the barges and fueling the towboat while en route have made it possible to increase the towboat's power without increasing its size, and to permit the boat to be operated at its most efficient draft.

8) Special weatherproof rolling covers have been developed for general cargo barges, which give complete protection to dry cargo yet permit quick opening and easy access for unloading by derrick or clamshell.

The effect which these new developments in towboat and barge design have had on waterborne commerce has been astounding. In the 10-year period from 1945 to 1955, the ton-miles of traffic handled on the inland waterways was 2.4 times that handled in the immediately preceding 10-year period from 1935 to 1945 and 5.5 times that handled in the 10-year period from 1925 to 1935.

Although recent progress has been remarkable, there will be much more progress to come. River channels need to be deeper. The speed of tows is greatly affected by the depth of water; and, as greater depths can be provided, great speeds can be maintained. The present 600' locks are not now adequate, and plans are under way to adopt 1200' as a standard length. This 1200' length has already become an accomplished fact in the proposed new locks for the Ohio River, some of which are now under construction. Much yet remains to be done in radar development. Bridge piers, buoys, and lock walls are difficult to distinguish by radar, and further developments will greatly facilitate its use. Better radio communications by the use of very high frequency channels will replace most of the short range communications.

Integrated units will be larger and longer, and the system of bow steering will come into more general use on all long tows.

A great and prosperous waterway transportation industry is taking shape

in the Mississippi Valley, and the savings are reflected in the entire price and freight structure of the Mississippi Basin.

This industry can prosper and grow only through better service and benefits to the public. Continued improvements in the efficiency and operation must be made so that the cost of transportation can be held to the absolute minimum. The savings thereby accomplished must be reflected in lower prices to the consumer, which will not only tend to raise the standard of living here at home, but it will also help the industries of the Mississippi Valley to be more competitive in world markets.

River Transportation has again taken its proper place in our American Way of Life, and this time we must not let it die. We must be forever alert to attempts at strangulation by restrictive regulations, tolls, or other means. We must keep our inland waterways forever free for the use of all the people.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

CURRENT TRENDS IN OHIO RIVER TRAFFIC AND EQUIPMENT[a]

Charles F. Michiels,[1] William F. Lail,[2] and Robert E. Mytinger[3]
(Proc. Paper 1123)

SYNOPSIS

This paper first presents a brief resume of the history of navigation on the Ohio River and explains how the pattern of traffic and the characteristics of available equipment affected the original plan of canalization of that river. Recent changes and current trends in traffic on the Ohio River and in the principal classes of river transportation equipment are discussed. The paper concludes with a brief description of the new plan of improved canalization of the Ohio River, and with a discussion of the manner in which current trends in traffic and equipment have affected the canalization plan and the design of the new navigation structures.

DISCUSSION

The trends of traffic and equipment on an inland waterway such as the Ohio River may seem at first glance to be of only academic interest to a group representing the profession of Civil Engineering. Such trends might be of vital concern to industrialists, or boat and barge builders. In what manner though are they of more than passing interest to Civil Engineers?

The answer is that the pattern of traffic and the nature of equipment used on a waterway are, or should be, major factors in overall planning of canalization of that waterway and in design and construction of the individual locks and dams. These factors have not always been given due consideration in the past and the result has been early inadequacy and rapid obsolescence of expensive navigation structures.

Note: Discussion open until May 1, 1957. Paper 1123 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 5, December, 1956.

[a]Presented at the Annual Meeting of the October, 1956, ASCE, Pittsburgh, Pa.

1. Chief, Operations Div., Ohio River Div., Corps of Engrs., U.S. Dept. of the Army, Cincinnati, Ohio.
2. Ohio River Div., Corps of Engrs., U.S. Dept. of the Army, Cincinnati, Ohio.
3. Ohio River Div., Corps of Engrs., U.S. Dept. of the Army, Cincinnati, Ohio.

The original canalization system on the Ohio River consisted of 51 low-lift dams with single locks 600 feet long and 110 feet wide. The original plan contemplated 54 locks and dams, but during the course of construction it was determined that three of them could be eliminated. The canalization of the Ohio under that plan was started in 1884 and was completed in 1929. With the exception of four replacement structures—Emsworth, Montgomery and Dashields just below Pittsburgh, together with Gallipolis above Huntington, the original structures are still in use.

Fortunately, on the Ohio River System the capacity of the locks has become inadequate; the obsolescence of the system has become apparent, and the economic life of the existing structures has been reached all at about the same time. This indicates that the engineers who planned the original system and designed and constructed the navigation structures were either very good engineers or remarkably lucky.

The then existing and prospective commerce on the Ohio River and the equipment for handling that commerce were major factors governing the original plan for canalization of the Ohio River and the design of the navigation structures.

The Ohio River is subject to great variation in river stages and for a great portion of each year there is sufficient depth of water in the natural river channel to accommodate craft with a draft of nine feet or more. The existing and prospective commerce at the time the original canalization was planned consisted for most part of coal and steel products. Practically all of this traffic moved downstream to points along the middle and lower Ohio and to points on the lower Mississippi as far south as New Orleans. There was no upbound commerce either existing or prospective at that time which was considered important enough to be a governing factor in the canalization scheme. Also, the towboats of that era were of limited horsepower and could not consistently handle large amounts of upstream traffic even if there had been a demand for such traffic at that time. In view of these conditions, the ideal plan for canalization of the Ohio River appeared to be a series of locks and dams which would provide a minimum channel depth of about nine feet during low water stages and which would permit the downbound traffic to take full advantage of the current on higher stages without encountering the delays inherent in locking operations. The relatively small horsepower of even the best sternwheel towboats of the era limited the size of the tows that could be handled safely during low water stages to about 11 barges with dimensions of 26' x 175'. Such tows could be accommodated in a lock 600' in length x 110' in width.

Based largely on these criteria, the original navigation structures were of a design unique in the United States, although they were similar in design to some of the structures in use on the European inland waterways. The dam consists essentially of a Chanoine weir with an average lift of about seven feet. The Chanoine weir is simply a series of movable wickets four feet in width, extending from the river side of the lock to an abutment on the opposite shore. Each wicket, which is constructed of wood, is mounted on a steel framework which is hinged to the foundation of the dam. When the wicket is in the raised position, it is held in a position inclined slightly downstream from the vertical by a steel prop attached to the framework of the wicket, with the lower end resting against a stop located sufficiently downstream so that the angle between the prop and the wicket is approximately 90°. The lower end of the wicket rests against the upstream side of a sill which

extends about one foot above the foundation, the top of which is at about the elevation of the natural bed of the river. Probably the best description of this type of dam from a non-technical standpoint was given by a woman sightseeing at one of the structures. She described it as being similar to a number of folding ironing boards setting endwise across the river.

Included within the length of the dam are one or more hydraulically operated discharge weirs, which are designed to compensate for small fluctuation in flow during the period the dams are in operation.

When water stages in any section of the river become low enough to prevent craft with a draft of nine feet from navigating in the natural channel, the wickets are raised to form a dam and the lock is placed in operation. Conversely, when water stages become sufficiently high to permit safe navigation in the natural channel, the wickets are lowered. In that state the only obstruction the dam presents to navigation and the flow of the river is the sill which extends only one foot above the natural bed of the river.

These navigation structures met all of the requirements for the navigation existing at the time of their completion, and have continued to meet them to a large extent until the past several years.

Within the past ten to fifteen years, and particularly since World War II, there has been a consistent but rapid change in both the traffic and the traffic pattern. Where the great bulk of the traffic once consisted of coal and steel products, the commodities now transported on the Ohio River System are almost too numerous to list. Among those carried in significant volume are petroleum products ranging from crude oil produced in Louisiana and Texas, to high octane gasoline produced at refineries located on the Ohio River; chemicals both dry and liquid, bound for the Kanawha Valley in the vicinity of Charleston, West Virginia; bauxite for the production of aluminum; asphalt; portland cement; sand and gravel; grain; automobiles; and the old standbys coal and steel. While now in the experimental or development stage, the transportation of South American iron ore from the Gulf to Pittsburgh by barge may assume significant proportions in the near future.

Where once nearly all of the traffic moved downstream, now the up and downstream movements are approximately equal. Most of the petroleum traffic is upbound. A large portion of the coal traffic is now also moving upstream due to the use of West Kentucky and Illinois coal in the huge steam-electric generating plants constructed on the Ohio River in recent years.

Along with the changes in types of cargo and the traffic pattern have come great increases in tonnage and ton miles. The original canalization system was designed on the expectation that the maximum tonnage that would be carried on the Ohio River in any one year would probably not exceed 15,000,000 tons. In 1929, the year the system was completed, the traffic amounted to 22,000,000 tons and 1,512,000,000 ton miles. Last year the traffic on the main stream of the Ohio River amounted to 71,460,000 tons with a total of 14,900,000,000 ton miles. Undoubtedly this traffic will continue to increase for many years to come unless the entire industrial pattern for the country as a whole undergoes a radical change.

Let us now consider how the changes and trends in the traffic picture have been accompanied by changes in the transportation equipment used on the river.

It is appropriate to preface this discussion of equipment used by the navigation industry on the Ohio River by mention of the fact that the Pittsburgh area is the birthplace and the most important present day center of that

industry. As early as 1765 this area was the center of activity for construction of the flatboats and keelboats by which the early settlers descended the Ohio and Mississippi to points as far distant as New Orleans. It was here in 1811 that the first steamboat to operate on the inland rivers was built. It was on the Monongahela River that towing, as we know it today, had its origin more than a century ago. It was the continued towing activity in this area during the first decades of this century, when activity elsewhere on the inland rivers had declined practically to zero, that provided the opportunity for continued development of equipment and techniques which became the standards for a revived inland river commerce. It was not surprising then that a great many of the important advances in development of river navigation equipment were made in this area, or that Pittsburgh continues to be one of the most important centers of research and development and of production of equipment for the navigation industry on all the inland rivers, as well as the inland and seaboard canals.

The equipment used by the navigation industry on the Ohio River is not of a distinctive regional character but is the same as that used throughout the Mississippi River System and, to a large extent, on all other inland waterways of the United States. In this brief paper it will be necessary to limit discussion of trends to the two principal classes of that equipment. Those are, of course, the barges in which cargoes are transported and the towboats which propel fleets of barges and provide living quarters for the operating personnel.

The current trends in that equipment are not of recent origin, having been discernible as trends for from 10 to 25 years. While far from static, the present situation is not one of rapid change or radical new developments. It is, rather, one of steady improvement of design in accord with sound basic principles, of cautious adoption of new auxiliary equipment as it becomes available and promises increased efficiency of operation, and of careful selection of equipment to best fill the specific requirements of each new towing venture.

After two decades of the most remarkable growth from the verge of extinction, the inland river navigation industry had attained a stable and important position in the national economy by 1940. At that time the design of the few types of barges in wide use on the Ohio River had become standardized to a considerable extent. Of all-steel construction fabricated by arc-welding they had short blunt rakes at each end. Little attention had been given to development of efficient barge forms because of the low towing speeds and the predominantly downstream movement of loaded barges. Since that time barge design has changed in three distinct aspects.

Most noticable perhaps to the casual observer has been the marked trend toward larger barges. In general, maximum barge size, consistent with limitations of locks, with requirements of the intended service, and with structural considerations is desirable in order to: (1) minimize construction cost per ton of cargo capacity; (2) expedite loading and unloading of barges at terminals and assembly of barges into tow fleets and; (3) reduce towing costs by facilitating provision of a hull form offering low resistance. The latter consideration increases in importance as towing speed is increased. A good example of the general trend is the increase in size of the open-hopper barges used for transport of coal and of many other commodities which do not require protection from the weather. The old "Pittsburgh standard" coal barge measured 26' x 175', those dimensions reflecting the limitations of the 56' x 360'

locks on the Monongahela River, and had a capacity of about 1,000 tons. That early barge, of which a great number continue to be used, was followed by the 35' x 195' so-called "jumbo" barge, those dimensions being a function of the 110' x 600' Ohio River locks. This size barge has a capacity of about 1500 tons and is the predominant size in current construction. About two years ago the first really large coal barge appeared. This is a 50' x 290' barge with a capacity of from 3200 to 3500 tons. This size barge is intended for long distance hauls at relatively high speed and approaches the maximum size practicable at this time in view of the limitations of existing locks, channels and terminal facilities. The general trend toward larger barges is indicated also by noting that since 1950 the number of dry cargo barges on the inland rivers has increased by 27 percent, while the total capacity has increased by 35 percent. Over the same period the number of tank barges has increased by only 8 percent, while the total capacity has increased by 19 percent, all these figures being approximate.

The second noteworthy trend in barge design during the past ten or twelve years has been the adoption of more efficient hull forms and fleet arrangements. Developed through scientific analyses and extensive model studies, the improved hull forms offer less resistance in towing without sacrifice in cargo capacity. Their use permits larger tows, or higher towing speeds, without increase in towboat power and thus directly reduces towing costs per ton mile. In the face of steeply rising costs of labor and equipment, the availability of more efficient barges has been a factor of great economic importance to the towing industry. The highest state of this development is represented by the so-called integrated fleet. In such a fleet the lead and trailing barges have rakes on one end only, the other end being "square". The remaining barges in the fleet are square on both ends. When assembled such a fleet presents smooth bottom and side surfaces for its full length. It has greater cargo capacity than a similar fleet of conventional barges and its resistance is about 25 percent lower. The efficiency of the integrated fleet is seriously impaired if the square end of a barge is exposed by loss of the leading or trailing unit. Accordingly, the fully integrated fleet finds its best application in long haul service where it remains intact from point of origin to destination. The semi-integrated fleet offers greater flexibility in operation and is more widely used. It is composed of pairs of barges, each barge having one raked and one square end. The efficiency is not quite as good as that of the fully integrated fleet, but the flexibility in adding barges or dropping off barges at intermediate points has substantial value in most towing operations. There is a definite current trend toward wider use of integrated and semi-integrated fleets in many trades. Thought is being given to standardization of barges to the point where the benefits of integration can be extended to the common carrier service where the tows consist of many different commodities requiring pick-up and delivery at many intermediate points along the carrier route.

The third principal trend in barge design in recent years has been the development of a wide variety of special-purpose barges. The deck-load barge for sand and gravel and general use in the heavy construction industry, the open-hopper barge previously mentioned, the covered hopper barge for cargoes requiring protection from the weather, and the tank barge for petroleum products and other liquid cargoes are the predominant types of course. But in addition thereto there are now many barges designed specifically for transport of such cargoes as acetic, hydrochloric and sulfuric acids, caustic

soda, ethyl ether, liquified butane and propane, ammonia, chlorine, molten sulfur, asphalt and tar, portland cement, paper and paper products, grain and automobiles. The exterior of the hulls of these special purpose barges are the same size and form as the dry cargo and tank barges mentioned in order that they can be incorporated in a tow without difficulty. The interior hull arrangements are as required for the product to be transported. The increasing use of river transportation by the chemical industry is clearly reflected in the above list of products. There is every reason to believe that the trend toward provision of special barges for the products of various new industries that are attracted to the rivers in the future will continue.

The twin-screw diesel powered towboat HERBERT HOOVER, built in 1931 for the Inland Waterways Corporation, was the forerunner of the modern powerful inland river towboats. She had two 1100-horsepower propelling engines driving 8-foot diameter propellers. Purchased by the Mississippi Valley Barge Line Company in 1948, and renamed the NEW ORLEANS, she was repowered with modern 1600-horsepower engines and is still an impressive sight on the rivers. By 1940 the general arrangement and exterior appearance of inland river towboats had become fairly well standardized. Since that time the principal changes in towboat design have been in technical detail rather than in general form with improved boat efficiency as the prime objective.

There has been a definite trend toward more powerful towboats. The introduction of the Kort nozzle in 1937 provided a means of increasing the effective push-power by as much as 25 percent without increase in engine horsepower. The use of Kort nozzles has now become so common that they are considered characteristic of the modern inland river towboat.

Towboat horsepower has increased also and the trend toward higher horsepower continues. Ten years ago 3200-horsepower represented the most powerful towboat. That rating is now common. Last year two twin-screw boats rated at 4200-horsepower were built in Pittsburgh. Two triple-screw boats rated at 4800-horsepower have been in regular service on the Ohio and Mississippi Rivers for the past five years. There are two quadruple-screw 3600-horsepower boats operating on the Missouri River. Nearing completion at this time in Nashville is a 5400-horsepower towboat intended to handle a tow composed of nine 50' x 290' coal barges. The efficient application of great horsepower in relatively shallow draft vessels imposes the requirement for refinement of hull lines. Such refinement, based on extensive model studies, has been achieved largely during the past ten years.

It could be said that the trend toward the use of diesel engines for towboat power continues. Actually, however, the diesel engine has predominated the towboat field to the practical exclusion of all other types of power for the last 15 years. There has been a continuing trend toward the use of relatively high speed non-reversing engines driving propellers through reverse and reduction gearing rather than the slow speed direct connected reversing engines which were the predominant type 20 years ago. This trend is based on the relatively low cost, small size and light weight of the high speed engine and the simplicity of remote control of such engines and their associated driving gears.

All towboats of recent construction provide for pilothouse control of the propelling engines. Such control is more dependable and far more precise than the pilothouse-to-engineroom telegraph systems employed on the steamboats and the earlier towboats. Another great benefit from pilothouse control

is that in eliminating requirement for constant attendance of the engine control panel, it permits towboat operation with a smaller engineroom crew.

All of the foregoing developments have contributed to improved towboat efficiency and, consequently, to lower ton mile costs by increasing the towboat's ability to push more tons faster and with smaller crews. The overall efficiency of towing operations has been increased further in recent years by the adoption and widespread use of auxiliary equipment such as powered towmakeup winches, radio, radar and automatic steering devices. The powered winches expedite the assembly of tows and the connection and disengagement of the towboat from its tow fleet. The radio insures continuous contact between the towboat pilot, the pilots of other boats in the vicinity, the personnel at the locks the towboat may be approaching, and the home office from which the towboat's activity is directed. Thus, the pilot is able to operate his tow in a safer and more efficient manner than would otherwise be possible. The radar permits towing to be continued during periods of low visibility due to fog, snow, smoke and rain, where without it towing would have to be suspended for indefinite periods. These developments thus increase the effective towing time with resultant increase in efficiency of operation.

There is a steady slow trend toward wider use of rate-of-swing indicators and automatic steering equipment. Such devices improve the efficiency of the towboat by instantly detecting deviations from the desired course, thus permitting correction by the pilot or automatically with a small rudder movement. This promotes increased towing efficiency since large rudder movements seriously reduce the effective push-power of a towboat.

The engineering of equipment used by the navigation industry on the inland rivers has been greatly advanced in recent years. The use of model studies, checked by full scale tests with careful analysis of actual operations over long periods, has taken much guesswork from planning and application of equipment. It appears certain that current trends will continue within the foreseeable future without dramatic change unless basic improvements in waterways or propelling equipment occur. The advent of deeper assured channels would usher in an era of still larger barges and still more powerful towboats and of improved efficiencies in all towing operations. The possibility of the gas turbine as a prime mover in towboats is recognized by all concerned in towboat design. While the early displacement of the diesel engine from its present predominant position does not appear likely, further development of the gas turbine may result in future towboats of greater power, smaller size and lower in cost with resultant further increase in the overall efficiency of inland river navigation.

As was previously mentioned, the original canalization plan for the Ohio River has become obsolete and the present system of locks is no longer adequate for today's traffic. Recognizing that serious situation, the Congress has authorized funds to start the replacement of the existing structures with new locks and dams in accordance with a new canalization plan. Three new locks and dams are now under construction and the fourth will be started this year.

It may be of interest to point out briefly how the recent and current trends in traffic and equipment affected the new canalization plan and the design of the new structures. Certainly those trends required some radical departures from past thinking if growth of the navigation industry was to keep pace with the industrial development of the country in general and of the Ohio Valley in particular.

The great amount and varied nature of commodities now transported on the river, and the increase in upbound traffic, greatly modifies the original conception that the locks and dams should be designed to operate only on low water stages. With the movement of traffic being approximately equal both upstream and downstream, the current of the river ceases to become an important advantage to the transportation companies. Where a current will aid the movement of downbound traffic, it will impede the movement of upbound traffic to an equal or greater extent.

Low-lift dams are also a disadvantage under the current and prospective traffic pattern because of the greater number of structures and lockages required throughout the length of the river.

The improvement in the transportation equipment now permits the handling of tows the size of which is limited only by the dimensions of the locks and the curvature of the channel. The present 600 feet by 110 feet locks require that double lockages be made of a great number of tows. This is a very time consuming operation when compared to a single lockage, because the power of the towboat can only be utilized on the second section of the lockage.

The general consensus of navigators appears to be that a tow about 110 feet in width and 1200 feet in length is the maximum size that can be efficiently handled consistently on the Ohio River as a whole. This limitation is due to the configuration of the stream rather than dimensions of the locks.

Based on these criteria, the design of the new replacement structures is of the following general pattern:

a) The dam will be a gated non-navigable structure. It will have a low fixed sill which, with the gates open, will permit maximum flood flows with practically no swell-head.

b) The main lock will be 110 feet wide by 1200 feet long and there will be an auxiliary lock 110 feet by 600 feet. Exhaustive studies are being made to locate the locks and dams so as to provide the best entrance and exit conditions. Model tests have been made to develop a lock filling and emptying system that will keep the time of those operations to a minimum.

c) The lift of the new structures will range from about 12 to 35 feet, as compared to an average lift of about 7 feet for the original structures. This increase in average lift will reduce the number of structures on the Ohio River to 21 as compared to 51 structures built under the original system.

So we see that the amount and character of existing and prospective traffic; the general traffic movement pattern; and the type of transportation equipment in use and in prospect have all played a major role in the planning and design of the new canalization scheme for the Ohio River. It is hoped that the engineers responsible for this work are as good or as lucky as their predecessors.

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1124)

Page

Developing Port Facilities on Houston's Ship Channel, by Frank H. Newnam, Jr. (Proc. Paper 965. Prior discussion: none. Discussion closed.)
by Austin E. Brant 1124-3
by Ersel G. Lantz 1124-8

Control of Rio Grande by United States and Mexico, by J. C. Bustamente and J. F. Friedkin. (Proc. Paper 975. Prior discussion: none. Discussion closed.)
Corrections 1124-11

Note: Paper 1124 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, WW 5, December, 1956.

Discussion of
"DEVELOPING PORT FACILITIES ON HOUSTON'S SHIP CHANNEL"

by Frank H. Newnam, Jr.
(Proc. Paper 965)

AUSTIN E. BRANT, JR.,[1] J.M. ASCE.—The author is to be congratulated on his concise presentation of the difficult subject of design criteria for general cargo facilities and on the detailed studies made. It is of interest to compare the criteria and estimated costs developed by Mr. Newnam with those determined during the engineering and economic survey made in 1953 for the Harris County-Houston Ship Channel Navigation District (The Port of Houston) by Tippetts-Abbett-McCarthy-Stratton (at that time, Knappen-Tippetts-Abbett-McCarthy). The primary purpose of that survey was the establishment of a master plan for the future development of the Port's physical facilities, and it included the determination of basic criteria for general cargo marine terminals at Houston.

Basis for Determination of Criteria

Any modern general cargo wharf must necessarily be designed as a compromise of the primary features proposed by steamship operators, stevedores, terminal operators, and railroad and trucking interests. The criteria recommended by Tippetts-Abbett-McCarthy-Abbett for general cargo facilities at Houston combine as many of the desired characteristics as possible, to provide an efficient marine terminal at reasonable cost. In the development of these criteria, particular importance was placed on the opinions of those who ultimately must bear responsibility for the operating efficiency of Houston's public marine terminals.

Type of Wharf

Early in the survey, it was concluded that marginal wharves would be the most suitable type for development along the Houston Ship Channel, since they permit vessels to dock and undock without blocking the Channel. Moreover, a marginal wharf with adjacent transit shed and supporting area usually is more efficient in operation than any other type wharf. A marginal wharf also has a high degree of flexibility under varying usages and permits effective utilization of the upland areas for warehouses, open storage, railroad yards and water-oriented industries.

1. Project Engr., Tippetts-Abbett-McCarthy-Stratton, Cons. Engrs., New York, N. Y.

Layout of Berths

It was believed also that general cargo ship terminals should be built with units of two covered berths separated by a single-berth open wharf. This arrangement obviates the necessity of providing wide clearances between individual berths, thus permitting the most economical utilization of bulkhead frontage, sheddage, and road and track construction, while still preserving independent rail and truck access to each berth. Under usual operating procedures, operations at the two berths would be separate. In the case of very large cargoes or in an emergency, it would be relatively simple to expand or contract the storage areas at adjacent berths.

As a result of an investigation of warehousing costs and requirements in the Port area, it was concluded that facilities for warehouse storage of waterborne general cargo should be provided at shipside. Warehouse space constructed integrally with the transit storage space would aid in reducing the congestion caused by cargoes delayed excessively in transit storage.

Summary of Desirable Characteristics of Covered General Cargo Ship Berths

The principal criteria developed during the survey are summarized in Table 1. For comparative purposes, those presented by Mr. Newnam for the marginal wharf layout are given also. The table shows that the major differences between the criteria given by Mr. Newnam and those presented in the Tippetts-Abbett-McCarthy-Stratton report are the elevation of the wharf apron, the number of tracks on the apron, and the size of the transit storage space.

The elevation of 25 feet above mean low water given for the wharf apron in the Tippetts-Abbett-McCarthy-Stratton report would be higher than any recorded high water in the Ship Channel. Moreover, it would permit the establishment of reasonably convenient, safe grades on the rail connections between the proposed development area and both the Clinton Division of the Port Terminal Railroad and the existing wharves on the north side of the Turning Basin. As the elevation of Wharf 16 is about 18 feet and the elevation of the mainline of the Railroad near Clinton Drive is about 42 feet, any significant change in the basic 25-foot elevation proposed for these wharves would render more difficult one or the other of the proposed rail connections. This would be of particular importance in relation to the rail connection to the Port Terminal Railroad which would serve as the primary means of rail access to the area. An appreciable reduction in the proposed apron elevation below 25 feet would result in a steepening of the grade on this connection and add measurably to the hazard of rail operation. Moreover, lower elevations would increase excavation costs, whereas higher elevations would increase the cost of the bulkhead structure. It is considered that, with the elevations chosen, an economic balance can be achieved.

Two tracks are proposed in the Tippetts-Abbett-McCarthy-Stratton report for the wharf apron since it is believed that one loading track and one running track is usually adequate for operations at a general cargo terminal.

Modern cargo vessels, such as the C4-S-A4 cargo class have a bale cubic capacity of up to 640,000 cubic feet. A study of the general cargo commodities stored in the covered transit sheds at Houston indicates that the average

TABLE 1

PRINCIPAL CRITERIA FOR COVERED GENERAL CARGO SHIP BERTHS

	Mr. Newnam's Paper	Tippetts-Abbett-McCarthy-Stratton Report
Berth:		
Length	600'	600'
Depth (below MLW)	40'	40'
Wharf Apron:		
Width	50'	40'
Elevation (above MLW)	20'	25'
Transit and Warehouse Storage:		
Net transit storage area, per berth	not given	85,000 sq. ft.
Net warehouse storage area, per berth	none	40,000 sq. ft.
Gross transit storage area, per berth	100,000 sq.ft.	117,500 sq. ft.
Gross warehouse storage area, per berth	none	57,500 sq. ft.
Length of shed, per berth	500'	500'
Width of shed, per berth	200'	350'
Width used for transit storage	200'	235'
Width used for warehouse storage	none	115'
Clear stacking height	20'	20'
Width of rear loading platform	20'	25'
Truck Facilities:		
Width of main terminal roadway	44'	48'
Width of feeder roadways	24'	24'
Maximum grade	not given	5%
Width of Truck Maneuvering area at rear of transit sheds	100'	125'
Railroad Facilities:		
Maximum permissible degree of curvature	not given	19.1°
Maximum desirable degree of curvature	12.5°	18°
Maximum grade	1%	1%
Holding yard capacity, per berth	33 cars	50 to 60 cars
Tracks on wharf apron	3	2

TABLE 2

ESTIMATED COSTS FOR CONSTRUCTION OF GENERAL CARGO WHARVES

IN AREA SOUTH OF PUBLIC GRAIN ELEVATOR

600—foot ship berth (covered)

1.	Wharf platform 60 feet wide and shed platform on fill 355 feet wide, including dredging, excavation, pavement on compacted fill, track wells, trackage, mooring facilities, and other appurtenances.	$1,130,000
2.	Shed superstructure (350 feet x 500 feet) and footings, semi-fireproof construction and sprinklers.	820,000
3.	Excavation and grading at rear of wharf proper for holding yard, roadways, truck pavements, parking and open storage areas.	90,000
4.	Railroad holding yard, including track leads extending to wharf proper.	60,000
5.	Roadways and truck pavements.	45,000
6.	Parking and open storage areas.	30,000
	Sub-Total	$2,175,000
7.	Engineering, legal, administration, and contingencies @ 20%	435,000
	TOTAL	$2,610,000

575—foot ship berth (open)

1.	Wharf platform 60 feet wide, 355-foot pavement on compacted fill, including dredging, excavation, trackage, mooring facilities, and other appurtenances.	$1,080,000
2.	Items 3, 4, 5 and 6 as above for covered ship berth.	230,000
	Sub-Total	$1,310,000
3.	Engineering, legal, administration, and contingencies @ 20%	260,000
	TOTAL	$1,570,000

weight of cargo handled is about 40 pounds per cubic foot. In the transit storage areas at the Port, it has been found economical to stack most general cargo three pallets high for an average floor loading of about 300 pounds per square foot of net storage area. The net transit storage area required to accommodate the contents of a general cargo vessel of 640,000-cubic foot capacity can be determined as follows:

$$640{,}000 \text{ cu. ft.} \times \frac{40 \text{ lbs./cu. ft.}}{300 \text{ lbs./sq. ft.}} = 85{,}000 \text{ sq. ft.}$$

This should be increased by about 40% to allow for aisles and sorting space, giving a gross transit shed space required of about 117,500 square feet.

Estimated Costs of Construction

Preliminary cost estimates for the construction of covered and open general cargo wharves in the area covered by Mr. Newnam's paper were made during the 1953 Tippetts-Abbett-McCarthy-Stratton survey. These estimates, which include the cost of all rail, roadway, and supporting facilities, are given in Table 2. Using those costs, the total estimated cost for the 15-berth development now proposed would be about $34,000,000. In comparing this cost estimate with that given by Mr. Newnam, allowances should be made for the increase in the cost of construction since 1953 and the different size and type of sheds.

The estimates given in the Tippetts-Abbett-McCarthy, Stratton report are considerably higher than the actual costs for wharves previously built in the Turning Basin area. These increased costs result from provisions to improve the functional layout and increase the operating capacity per berth, chief of which are the following:

1. 75-100% more covered storage than at existing berths—to provide sufficient area to handle not only the largest foreseeable requirements for transit storage, but also a considerable amount of shipside warehousing.
2. Semi-fireproof construction of sheds and the provision of sprinklers—required for transit and warehouse operations under same roof.
3. Railroad holding yard for 50 to 60 cars per berth—required in view of already saturated condition of the Port Terminal Railroad's North Yard, and because of the distance (average in excess of two miles) between North Yard and the proposed wharves.
4. Independent roadway network—because the existing roads in the Turning Basin area would be inadequate for additional traffic to or from the Endowment Area.
5. Considerable earth moving and grading required for the development of adequate back-up areas and for the new rail and road facilities.

CONCLUSIONS

The criteria and cost estimates presented by Mr. Newnam for the construction of marine terminal facilities at Houston agree in general with those developed in the 1953 master plan survey made for the Port by Tippetts-

Abbett-McCarthy-Stratton. The substantial agreement between these two studies should encourage the Port of Houston in the development of additional marine terminal facilities to insure the future growth of the Port and the maintenance of its standing as one of the major ports of the nation.

ERSEL G. LANTZ, A.M. ASCE.*—The interesting paper by Frank H. Newnam, Jr. presents a number of valuable criteria to use in Port Development. The tables containing the answers from the railroads, trucking lines, steamship lines, and stevedoring companies illustrate the wide difference of opinion among the users of Port facilities. The rapid growth of the use of mechanical handling equipment, especially fork trucks, during the past few years has raised problems in port design that the builders of the older docks, sheds, and wharves did not and could not foresee.

The comparative small Port of Brownsville just completed (October, 1956), a marginal type $3,500,000 dock and shed facility. The following table compares the Port Brownsville facility with Table 1 BASIC DIMENSIONS AND CRITERIA ADOPTED as shown on page 12 of the paper.

Under Item 1 this Port and the writer does not believe that the 20 feet of apron height can always be justified. One advantage would be in the use of shipside cranes operating from a railroad track, inasmuch as the high dock would give the crane operator a better view in handling cargo into or out of the hold. The wide variety of sizes and loaded conditions prevailing among the ships operating out of any port that does not handle a specialized cargo makes the distance traveled by the load so variable that any savings as a result of the shorter distance would be nebulous. Many of the small freighters and banana boats operating into the Port of Brownsville would have trouble operating over a dock with 20 feet elevation. The expense added to the bulkhead cost by the extra 8 feet of height far outweighs any known advantages.

Port Brownsville used an apron width of 50 feet for the new dock because past experience with 35 and 40 feet width of apron had shown the need for wider aprons in giving more maneuverability to equipment; and space for the temporary storage of dunnage, ship supplies, and cargo. The extra width of the apron causes the cargo to travel farther on its way to the hold of the ship, but the advantages gained by the extra working space provided by the wider apron in stacking and classifying cargo and for storing dunnage far outweighs the extra expense caused by the longer travel.

Port Brownsville's new shed width is 200 feet. This width is greater than Longshoremen will move cotton with hand trucks without an extra handling charge; however, the Port felt that the extra space could be used for cotton storage during the peak season. In handling general cargo or cotton by mechanical equipment the extra longshoreman charge does not apply. The Port uses 500 feet of shed space for each ship's berth; however, the 1,000 feet of shed on the new dock is divided by firewalls into three sections. This was done to satisfy an insurance requirement that will not permit more than 7500 bales of cotton in one fire division without penalty.

The clear height of the shed at Port Brownsville is about the same as that recommended in the criteria. This is a minimum shed clearance with the present day use of fork machines for the stacking of cotton and pallettized cargo. The Port built the rear platform 30 feet in width to permit the unloading of the cotton trucks on the apron outside the shed because cotton insurance

* Engr., Port Brownsville, Brownsville, Tex.

Item:	Dimension or Criteria:	Port Brownsville Docks 10-14:
1. Wharf Apron:		
Height above M.L.W.	20'	12'
Width	50'	50'
2. Transit Shed:		
Width	200'	200'
Length per berth	500'	500'
Clear height (bottom of roof truss to floor)	20'	19'
Width of rear loading platform	20'	30'
Width of canopy, rear loading platform (extended to centerline of first track)	28'6"	none
3. Width of Paved Loading Area Back of Shed Marginal Wharves	100'	100'
4. Length per Berth (marginal type)	600'	600'
5. Single Slip Length (pier type)	800'	N.A.*
6. Width of Slip (pier type)	300'	N.A.*
7. Railroads:		
General:		
Minimum track spacing	13'	14'
Maximum degree of curvature (where economically possible)	12°-30'	18°
Maximum grade (where economically possible)	1.0%	1.5%
Storage yard capacity	500 cars	200 cars
Length of storage yard	50 car lengths	25 car lengths
Marginal Type:		
Number of tracks, wharf apron	3	2
Number of tracks, behind shed	3	2
Double RR crossovers, wharf apron between spotting tracks	1 at center of each berth	1 at center of dock area
Single RR crossovers, behind shed (independent switching of cars at each berth)	1 at each end of shed	1 for three sheds
8. Roadways:		
Main road width	44'	26' paved
Access road width	24'	30' paved
9. Design Depth of Water at Fender Line	-40'	-32'

*N.A. - Not applicable

requirements will not permit the entry of trucks into the sheds. In the climate prevailing around the Port area a canopy over the rear apron and track was not felt to be economically justified and cotton warehouse regulations do not permit the canopy without penalty.

Item 3: Port Brownsville is providing 100 feet of paved unloading space in the back of the shed to permit trucks to back up and unload onto the rear apron.

Item 4: The 600 feet of berthing space provided is ample. Port Brownsville has extra space at each end of the shedded area to give approximately 600 lineal feet per berth.

Item 7: Port Brownsville set a minimum track spacing of 14 feet. The maximum degree of curvature was set at 18 degrees; however, local conditions caused some curves to be 22 degrees and 30 minutes. Special gauge rods were installed on all curves, and, so far, even the extreme curves have given no trouble. The maximum grade is set at 1.5 percent in the tracks installed for the new docks. Present track storage is only 50 cars; however, space has been left for future tracks if needed. Port Brownsville does not feel that a third track on the apron would be worth the extra cost involved.

Item 8: The road sizes need no comment except to say that extra space is provided on Port Brownsville's access road for parking trucks awaiting their turn to go onto the dock. Parking for dock personnel is provided in back of the loading area.

Item 9: The design depth of the dock, 32 feet, is considered sufficient for many years to come; however, special provision is made to increase this should it ever become necessary.

Table II: The only item that can be compared here is the cost per berth less sheds, the last item in the table.

Port Brownsville's new dock unit provides three deepsea berths and one 20 foot deep berth which can be converted to a deepsea 32 foot dock for $2,900,000, and a future $60,000 dredging cost would give four deepsea berths for a cost of $2,960,000, or $740,000 per berth.

The writer believes a great deal of the extra $1,360,000 per berth shown in the table is due to the greater height of the apron as well as to the greater design depth of the channel.

Mr. Frank H. Newnam, Jr., is to be complimented on an excellent paper and he has added a great deal to the somewhat limited design criteria available to port personnel.

Discussion of
"CONTROL OF RIO GRANDE BY UNITED STATES AND MEXICO"

by J. C. Bustamente and J. F. Friedkin
(Proc. Paper 975)

CORRECTIONS.—On page 975-1 the date 1948 in both the Synopsis and in the first paragraph of the paper should be changed to 1848.

Paper 1128 WW 5

Journal of the
WATERWAYS AND HARBORS DIVISION
Proceedings of the American Society of Civil Engineers

LOCAL FLOOD-PROTECTION PROJECTS, OHIO RIVER BASIN

S. M. Bailey,[1] M. ASCE, R. E. Karlen,[2] A.M. ASCE,
and Harry Pockras[3]
(Proc. Paper 1128)

SYNOPSIS

Local flood protection projects consist of levees, concrete walls, traffic closures, interior drainage facilities, pumping plants, and channel improvements. General requirements for planning and design of the various parts of the projects are presented in this paper. Information is included as to the effectiveness of one of the projects during a flood.

The great flood of 1937 on the Ohio River, (The maximum of record from Point Pleasant, W. Va. to the Ohio Mouth, a distance of 719 miles) together with the 1936 flood (the maximum of record in the Pittsburgh, Pa. area), aroused the citizens of the affected localities to the realization that comprehensive measures would have to be undertaken to avoid similar catastrophies in the future. This manifested interest in flood relief, both in the Ohio River Basin and elsewhere, led to the enactment by Congress of the Flood Control Acts of 1936, 1937 and 1938.

These acts authorized the Corps of Engineers, U. S. Army to undertake a flood control program in the Ohio River basin which included both local protection projects and reservoirs. Later acts provided funds to the Corps of Engineers for planning, construction and supervision of these projects. Local protection projects are complete units in themselves, consisting of earth levees, concrete floodwalls, pumping plants, drainage facilities, closure structures and their necessary appurtenances, and protect only the specific population center for which designed.

Note: Discussion open until May 1, 1957. Paper 1128 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, No. WW 5, December, 1956.

1. Chief, Eng. Div., Corps of Engrs., U. S. Dept. of the Army, Louisville, Ky.
2. Engr., Corps of Engrs., U. S. Dept. of the Army, Louisville, Ky.
3. Chief, Eng. Div., Huntington Dist., Corps of Engrs., U. S. Dept. of the Army, Huntington, W. Va.

With certain minor exceptions, the following cooperation, as specified in Acts of Congress, must be provided by local interests for local protection projects which involve levees and flood walls:

a) Provide without cost to the United States all lands, easements, and rights-of-way necessary for the construction of the project.

b) Hold and save the United States free from damages due to the construction works.

c) Maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of the Army.

Planning of Levees and Walls

The first step in the planning of a local protection project is the determination of the alignment and the type of construction for the proposed project. Topographic maps are used for layout of the protection line, the scale being dependent upon the development of the area.

It is evident that the most economical type of construction from the point of view of the Government may not result in the least overall cost when the expense to the Government and local interests are considered together. In general, the least costly type of construction to the Federal Government will be an earth levee. However through a well-developed or industrial area, the costs of obtaining the necessary rights-of-way for an earth levee may be prohibitive. Conversely, the least costly type of construction from the point of view of local interests will be a concrete wall because of the lesser widths of rights-of-way required. The concrete wall, however, will cost several times the amount required to construct a levee of equal height, and the overall cost of the project may be so large that the project may not be economically feasible.

Height of Protection

After the tentative alignment is established the next step is the determination of the height of protection required. The establishment of the design grade for local protection projects along the Ohio River and some of the tributaries is intimately related with the system of reservoirs included in the comprehensive plan. A project flood for the Ohio River has been established on the basis of the maximum anticipated flood resulting from a combination of the worst general storm occurring in various regions of the basin. The flood height, as estimated to be reduced by the reservoirs included in the plan, is generally below the natural maximum crest profile. In general, the top grade has been designed 3 feet above the maximum flood of record as modified to compensate for confinement by the protection works on each side of the river. Design grades for local protection projects on tributary streams are in general somewhat less in height in respect to potential floods without undue hazard.

Selection of Types of Protection Structures

Selection of the type of structure to furnish protection at a particular point is based on comparison of levee structure plus levee rights-of-way costs and

wall structure plus wall rights-of-way costs. On the average, levee structure costs are less than one-fifth those of walls, but the levee requires about three times as much right-of-way. These inherent conditions ordinarily result in the outlying less developed areas being protected by levees while the highly developed urban areas are protected by walls. After the general location of the protection line is established, foundation conditions are determined by extensive investigations with auger borings, drill holes and test pits.

Levee Design

A typical levee cross-section under normal conditions has a crown width of 8 feet to 12 feet and has 1 on 2-1/2 or 1 on 3 side slopes. Exhibit No. 1 shows a typical section of a levee. Where the height of levee exceeds 25 feet or where foundation conditions are inadequate the slopes are flattened as required to provide a stable structure. Unsuitable foundation materials are removed. Some levees have been built to a height of 55 to 60 feet. The embankment material is placed in approximately 8 inch layers and compacted at optimum moisture content by not less than six passes of sheepsfoot rollers. On levees protecting agricultural areas or other low types of developments, the material may be compacted at natural moisture content by means of hauling and spreading equipment, supplemented by rolling where necessary. An inspection trench is excavated to a reasonable depth in order to determine the location and condition of active or abandoned pipes or sewers under the levee that may impair the safety of the structure. Where the impervious foundation blanket is relatively thin, a relief drain or system of relief wells is placed at the landside toe to aid in controlling the seepage through the foundation and in draining seepage water to a point of collection. The drain also serves to stabilize the landside slope of high levees by concentrating the flowlines of seepage water within the levee. Where the levee is likely to be scoured by river currents the surface is protected by stone riprap.

Types of Walls

As previously mentioned, if, in the built-up areas, rights-of-way are too expensive to permit earth levee construction, concrete wall is used. The type of wall most generally selected for protection heights over ten feet is the reinforced concrete inverted "T" wall with a base width approximately equal to its height. Exhibit No. 2 shows various types of flood walls. For walls up to 20 feet in height a flat base is normally used. However, for walls in excess of 20 feet a sloping base generally is more economical. The base is buried three or more feet underground and is provided with a cut-off key at the riverside edge to prevent sliding and to increase the seepage path. Stability of this cantilever type wall is assured by the wide base and riverside cut-off key. Sloping base walls have been built to a height of 35 feet at Parkersburg, W. Va. The criteria for design of these walls are as follows:

a) A water load to the top of the structure is used for determining stability from overturning.

b) A factor of safety against sliding of not less than 1.5 is required when shear strength of the foundation only are considered. The method of analysis used is an adaptation of the Swedish Circle.

c) A creep ratio of not less than 2.0 is required for the types of foundation materials normally encountered in the Ohio River basin.

Structurally the wall is proportioned so that the unit stresses under the various design loadings will not exceed 18,000# sq. in. for reinforcing steel and 1,050# sq. in. for concrete in compression.

A second type of wall that is used for lower protection heights is known as the "I" type. Although this wall has been used for hiehgts up to 15 feet, current practice limits the clear height of this type to 10 feet. For economic reasons, two forms of this wall have been installed. For heights up to seven feet above the ground, the wall is constructed entirely of concrete with the portion embedded in the ground equal to approximately 1.35 times its clear height above the ground. For heights of seven to ten feet above ground, the wall consists of steel sheet piling, driven to the depth necessary to provide the required penetration ratio and topped with a reinforced concrete cap which extends four feet below the ground surface. On the steel sheet pile I-wall, a small concrete toe is added on the landside to reduce the deflection under load and to retain the earth adjacent to the wall. This wall is designed for water to the top with a penetration ratio of 1.35 to 1 in foundation materials having an angle of internal friction of ϕ equal 0.5, and a maximum moment of 29.35 h^3 at 0.67 h below the ground surface where h is the net protection height.

Along the Licking River front at Newport, Ky., an unusual design was required. The bank slopes of the river in this area are steep and unstable, with the banks rising on slopes between 1 on 3 and 1 on 2-1/2 from the water's edge to a height of approximately 50 feet. The buildings of the Newport Rolling Mill are constructed along the top of the bank which necessitated that any flood wall would have to be constructed within the slope area. There is evidence that the present bank, consisting of about 30 feet of debris and 20 feet of silty clay overlying sand, is inherently unstable. A typical "T" wall constructed on a bench of the slope would not strengthen the bank but might actually weaken it, since the necessary impervious cutoff wall at the heel would trap water landward of the wall and thus increase the drawdown driving forces. An investigation was made of a cellular steel piling wall with the piling driven into the underlying sand strata for stability. The design provided for a wall of interconnected cells filled with sand and gravel. The landside and riverside faces of the wall consist of a series of arcs connected by straight diaphragm walls similar to that used in one common type of cofferdam construction. The maximum thickness of the wall (arc to arc) is about 65 feet and the maximum distance between diaphragms is 38 feet. As it was unnecessary to carry the full thickness of the wall to protection grade, the top elevation of the landside portion was set at top of bank level. Protection grade was attained by using longer piling in the riverside portion of the wall and setting in an auxiliary arc of piling about 20 feet back of the riverside face, thus forming an auxiliary cell 20 feet high on top of the main cell with three walls common to the main cell. The length of piling required was 73 feet for that portion of the main cell landward of the auxiliary arc and 93 feet riverward of the arc. The piling length for the auxiliary arc itself was 33 feet. Exhibit No. 3 is a diagrammatic sketch of this wall.

A drainage system at the landside toe to control seepage is installed for all types of walls, except for those of extremely low heights. This system consists of a pipe relief drain for most conditions, with relief wells being

used for extremely pervious foundations. Walls can be constructed only on firm foundations. If excessive settlement of the natural ground is anticipated, the existing foundation materials are stripped and replaced with compacted material that will not be subject to excessive subsequent consolidation. If the extent of unsuitable foundation materials is such that stripping is not practicable, the wall is supported by bearing piles and a steel sheet piling cut-off is used. The walls are constructed in monoliths approximately twenty feet in length with water-tightness assured by a waterstop at each joint. Although a copper V-strip has been used extensively for this waterstop, current practice requires use of rubber strips of various designs. Tests have indicated that these rubber stops have a greater capacity to withstand differential movements without rupturing.

Ramps and Closures

The line of protection must of necessity cross roads, streets, and railroads. If a road or street crosses a levee in an area that is not too highly developed, it may be possible to ramp the road over the top of the levee. To reduce the road grade, sometimes the ramp grade is set three feet below the crown of the levee at their intersection. This depression in the levee is filled in with sandbags if the height of the flood so requires. Where a street or railroad crosses a wall, the surrounding development or ruling grade usually will prohibit raising the grade of the street or railroad to the elevation of the top of the wall. In these instances, recourse is made to the use of closure structures, which are openings in the wall to serve traffic during non-flood seasons.

During flood periods the openings are closed by gates, panels, or other means. There are two types of standard closures. One type consists of wood or aluminum stop logs placed on top of each other in vertical recesses provided for them in the abutments on each side of the opening. Where the width of the opening is in excess of 8 feet, intermediate steel or aluminum trestles are provided for installation in the sill anchorages, the number of trestles depending upon the width of the opening. The stop logs, generally 8 feet long, are then dropped into the vertical recesses in the abutments and vertical member of the trestle or trestles. Hold-down bolts are provided to secure a tight closure. This type of closure is shown on Exhibit No. 4. The other type of standard closure consists of light steel or aluminum trusses supporting a diaphragm of corrugated aluminum sheets, as shown on Exhibit No. 5. Each member is light enough so that it can be assembled by two or three men without power equipment. The trusses generally have a batter on the waterside approximately 30 degrees from the vertical. Metal purlins extend between the trusses and slots in the abutments on each side. The aluminum sheets are placed on the purlins with the corrugations running vertically. Adjacent sheets are joined by a continuous extruded clip joint acting as a seal. At the abutment slots, watertight joints are obtained by wooden wedges adjusted in the slots. The bottom seal is either made in a trench filled with mastic or by sandbagging. A later design of this type retains the trusses and purlins described but replaces the corrugated sheet with panels consisting of flat sheets reinforced with channel and modified I shapes. This fabrication results in a shallow pan. These panels are in two sections for the higher closures, as shown on Exhibit No. 6, permitting the installation of only the lower section

for moderate floods. When not in use, the gates or closure parts are housed in a concrete or masonry structure built integrally with, or adjacent to, the protection works.

Special types of closures have been used at some locations for structural or operational reasons. One type used for the Market Street closure at Jeffersonville, Indiana, shown on Exhibit No. 7, consists of a rolling gate 40 feet long and 15 feet high. Another type used for closure of an opening for the main line tracks of the New York Central Railroad at Cincinnati, Ohio, shown on Exhibit No. 8, consists of two hinged swing gates, each 21 feet long and 30 feet high. A third type used for closure of an underpass at the Baltimore and Ohio Railroad at Parkersburg, W. Va. consists of a steel lift gate 21 feet wide and 11 feet high, as shown on Exhibit No. 9. These type gates can be closed in a few minutes thereby permitting use of the openings for longer periods of time when flood stages are approaching the sill elevations.

Interior Drainage

Drainage facilities for local flood protection projects are of two major classifications: Those required to dispose of runoff during low river stages and those required for disposal during flood periods. In general, facilities in the first category consist of openings through the flood barrier designed to discharge all the flow that will reach the barrier without damage for design storm runoff and without excessive damage for storms of greater intensity. A typical surface drainage structure is shown on Exhibit No. 10. The flood period facilities consist primarily of ponding areas, pumping plants, and diversion ditches or sewers. In some cases, ponding areas alone will adequately serve an area during flood periods. Most areas, however, require pumping. At some locations where pumping is necessary, enough ponding may be utilized to greatly reduce the pumping rate. At other locations the removal rate must equal the rate of inflow to the pumping plant sump. Diversion ditches or sewers are constructed where it is less expensive to provide a combination of diversions and a single plant for two or more adjacent drainage areas than to provide separate pumping plants.

Basically, design of interior drainage facilities for the flood protection project deals with drainage as it exists at the site of the flood barrier and does not involve improvement of existing facilities. However, where local interests find it to their advantage to combine proposed sewage or storm water drainage facilities with flood protection works, coordinated construction programs are effected. In the cases of the Louisville, Ky., Newport, Ky., and Maysville, Ky. Projects, considerable portions of the drainage facilities were coordinated with sewage collection facilities planned by the Sewer Districts, thereby resulting in considerable savings in construction costs of both systems.

Outlet sewers provided at Federal Government expense are designed to carry flows consistent with design capacities used by the cities for their sewer construction. In Louisville, the 15-year frequency all-season storm is the basis for gravity drainage design in the urban area and provision is made to pass 100-year storm runoff without excessive damage. In general, the flood period pumping and drainage facilities are designed for runoff from a storm having 30-year frequency of occurrence coincident with Ohio River stages requiring pumping. Capacities of the pumping plants at the river

elevations which require initiation of pumping operations provide for rainfalls ranging from 0.5 to about one inch per hour, while the capacities at project design river stage correspond to a rainfall of 0.4 inch per hour.

Storm runoff rates for areas drained by natural channels are estimated by unit hydrograph application with runoff ranging from 30 percent of rainfall for some of the areas during all-season storms to 80 percent for some areas during flood periods. Rates of runoff from sewered areas are computed by the Rational Method with runoff coefficients varying from 0.4 to 0.9 for all-season storms and 0.7 to 0.9 for flood period storms. In some cases it is feasible to check Rational Method estimates by measuring sewer flows caused by isolated rainfalls. Salt-velocity measurements of flow have been made in some cases for determination of rating curves of main sewers and these rating curves were used to develop discharge hydrographs from which unit hydrographs were determined. Stage hydrographs are recorded on small 8-day water stage recorders installed in pipe wells located in manholes. Salt-velocity measurements also afford a check on coefficients of roughness estimated for application of the Rational Method.

Pumping Plants

Pumping plants usually are divided into two major types; the pond storage type and the closed conduit type, as shown on Exhibit No. 11.

The pond storage type plant is normally used for internal storm water removal only and is located on a natural surface drainage channel. The wet sump is usually open to channel flows and individual pump discharge lines are carried over the protection works to a common discharge structure on the riverside. A minimum of three equal size mixed or axial flow pumps, with a total capacity equal to the design capacity of the station, is generally provided. When the required capacity is small a centrifugal type pump is used. As sump storage is not required in this type station, provision is made for minimum sump area necessary to permit adequate spacing of pumps and installation of piping, unless increased size is necessary in the superstructure for installation of motors, controls, and other equipment. A circle of influence three times the suction bell diameter is used in determining the sump area, except where pump units are separated by walls, in which case the spacing is reduced to two diameters. The depth of the sump is governed by the minimum operating water level and adequate pump submergence to prevent detrimental vortex formation and cavitation. Minimum operating cycles of 30 minutes per pump are provided, and automatic safety shutoffs are included.

The closed conduit type plant is used to remove storm water, sewage and industrial wastes, and is located adjacent to the sewer. The sump is connected to the sewer by means of a gated intake and the pumps either discharge into a common chamber with a single pressure outlet through the protection line or have individual discharge lines over the levee to a common discharge structure on the riverside. In some cases, the flow is discharged back into the existing conduit on the riverside. In this type of station very little storage is available, so pumping capacity is provided to handle the design storm with only sufficient storage provided for practical operation of the pumps. In most cases either a common wet sump or modified dry sump is used. The modified dry sump is arranged so the area for the sanitary pumps is separated from the storm water sump by a weir. The sanitary pumps are

located in the storm water pit with intake pipes extending into the sanitary sump. When combined sanitary and storm flows exceed the capacity of the sanitary pumps, the flow will overtop the weir between the sumps and the storm water pumps will go into operation. Centrifugal pumps are normally used for sanitary flows, and for storm flows three or more equal size axial or mixed flow pumps of a total capacity equal to the design storm are provided. The closed conduit type station is generally fully automatic; that is, equipped to start and stop the pumps at predetermined water levels. The minimum operating cycle for these pumps is four minutes.

The pumps are driven by means of vertical shaft directly connected electric motors, which are controlled by switchgear within the plant. A cutaway view of a typical plant is shown on Exhibit No. 12. Squirrel cage type induction motors are used, except on the extremely large units, where synchronous motors are employed. A substation is constructed adjacent to the plant, and is energized only during flood periods under a special contract with the utility company.

The pumping plant structures are constructed of reinforced concrete below the motor floor, with brick superstructures in most cases, except when the plant is built integrally with the flood wall, in which case the entire plant is of reinforced concrete construction. Windows are omitted and other openings are kept to a minimum. The moist conditions to which the plants are subjected are extremely conducive to corrosion, with possible serious damage to motors and control equipment. Many of the superstructures of plants recently constructed have been dehumidified by a system consisting of vapor proofing the interior surfaces and removal of moisture by means of an activated agent with mechanical means for reactivation.

Where discharge lines are carried over a levee, they are extended down the riverside slope and terminated not less than 1-1/2 feet below the minimum river elevation for plant operation. This type installation assures siphon action thereby reducing operating heads on the pumps. In order to prevent reverse siphoning when the pump is shut off, a siphon breaker is provided at the crest of the discharge pipe. Where pumps discharge into a common chamber connected to a pressure conduit, the pump discharges are equipped with automatic flap gates to prevent reversal of flow. Pumping plants have varied from extremely small installations located in manholes below ground level to extremely large plants such as the Mill Creek plant at Cincinnati, Ohio and Beargrass Creek plant at Louisville, Kentucky. The Mill Creek plant consists of 6 units of 1875 c.f.s. capacity. The Beargrass Creek plant consists of 6 units of 960 c.f.s., one at 740 c.f.s. and a small unit of 160 c.f.s. capacity.

Channel Improvement

The type of local protection projects described above predominate along the Ohio River with some levee and wall projects along the tributary streams. Also on the tributaries are a number of channel improvement projects. These vary in extent from simple channel cleaning, rectification and enlargement (with or without paved slopes, and walls and levees to contain flood flows) to extremely complex projects. Many require numerous cutoffs or diversions. The more complex projects require relocation of roads, streets and railroads accompanied by alteration of existing bridges and the

construction of new bridges. They also require major utility changes and relocations, such as large water mains, gas mains, sewers and interior drainage facilities. A typical project of major complexity is the Massillon, Ohio improvement as shown on Exhibit No. 13. Whenever flood catastrophies are discussed invariably the great flood disaster of Johnstown, Pa. in 1889, which resulted in the loss of 2200 lives, is brought to mind. Johnstown was again visited with a flood in 1936, resulting in damage approximating $40,000,000. Exhibits No. 14 and 15 are views before and after improvements completed in 1943. It is estimated that the flood produced by Hurricane Hazel in October 1954 would have caused damages in excess of $22,000,000 had not the channel been improved. The stage of this 1954 flood was reduced about 12 feet by the improvement and all damages were averted.

Status of Program

Floods have occurred on the Ohio River since some of the local protection projects have been completed, and it is estimated that losses prevented at a number of the projects have more than paid off the original investments. Exhibit No. 16 shows pictorially a flood protection project in operation during a flood.

Under the comprehensive flood control plan for the Ohio River basin 242 local protection projects are authorized. To date 43 of the projects have been completed and 13 are under construction. Further studies are required, when funds become available, on many of the remaining projects to determine the economic feasibility of construction under present day conditions.

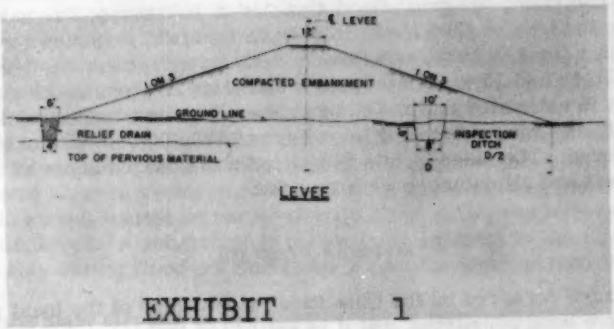


EXHIBIT 1

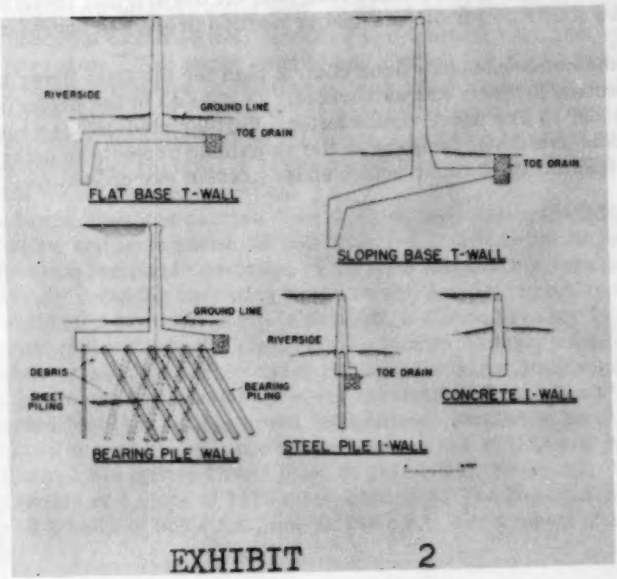


EXHIBIT 2

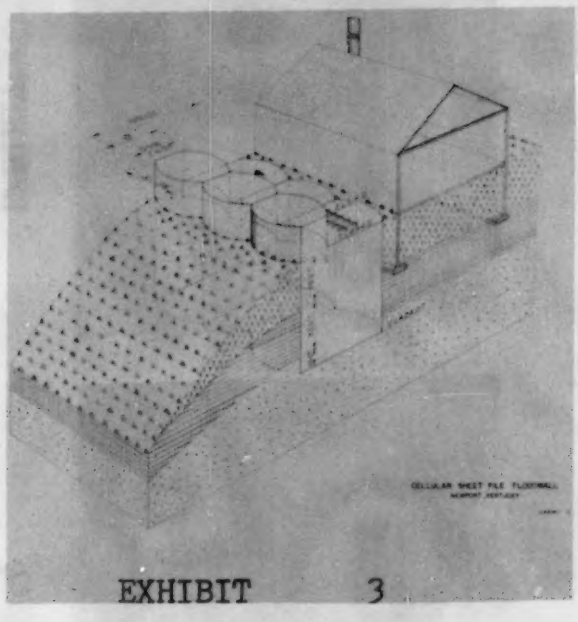


EXHIBIT 3

EXHIBIT 5

EXHIBIT 4

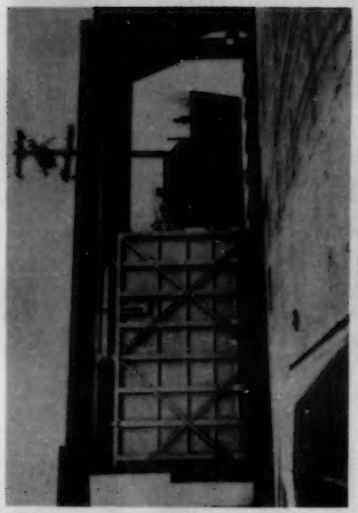
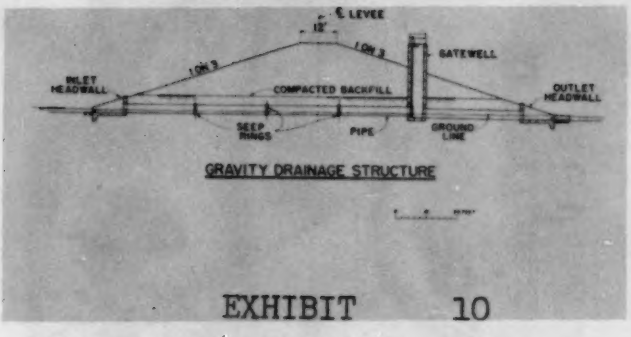


EXHIBIT 7

EXHIBIT 6

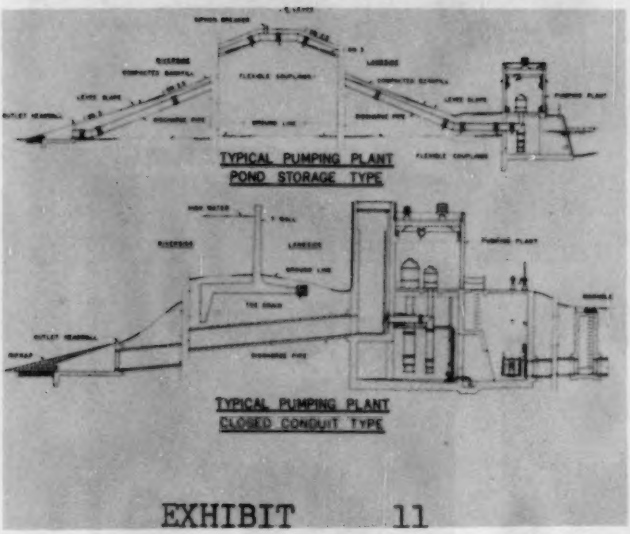
EXHIBIT 9

EXHIBIT 8



GRAVITY DRAINAGE STRUCTURE

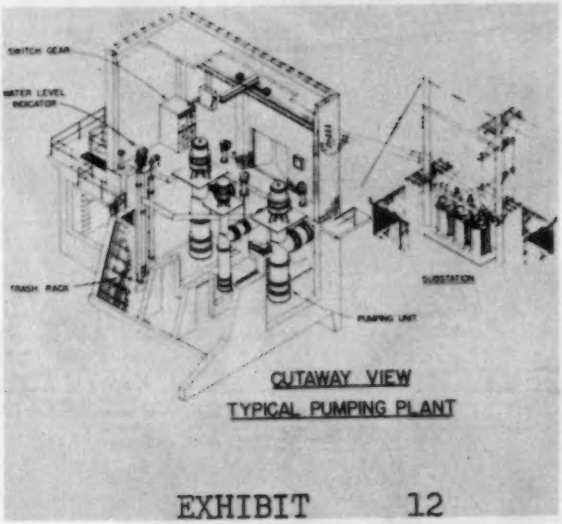
EXHIBIT 10



TYPICAL PUMPING PLANT
POND STORAGE TYPE

TYPICAL PUMPING PLANT
CLOSED CONDUIT TYPE

EXHIBIT 11



CUTAWAY VIEW
TYPICAL PUMPING PLANT

EXHIBIT 12

EXHIBIT 14

EXHIBIT 13

EXHIBIT 16

EXHIBIT 15

DIVISION ACTIVITIES

WATERWAYS AND HARBORS DIVISION

Proceedings of the American Society of Civil Engineers

NEWSLETTER

December, 1956

New Division Officers Named

Capt. Lewis C. Coxe, U.S. Naval Academy has been selected to head the Executive Committee of the Waterways and Harbors Division for 1956-1957. Members of the Executive Committee include Col. Rufus W. Putnam (Vice Chairman), Roger H. Gilman, Col. Charles M. Wellons and Evan W. Vaughan (Secretary). Col. Putnam is Executive Officer, State Lands Commission. Mr. Gilman is Director of Port Development for the Port of New York Authority. Both Col. Wellons and Mr. Vaughan are Consulting Engineers, with offices in Pittsburgh and New York respectively. These officers were selected during a meeting of the Executive Committee of the Division in Pittsburgh on October 17, coincident with the Annual Convention of the Society.

Chairman Named for Committee on Publications

Col. Ellsworth I. Davis, Chief of Staff, The Engineer Center, Hqs., Fort Belvoir, Virginia, has been selected by the Executive Committee of the Division to head the Committee on Publications. This committee serves as the outlet for all printed information arising within the Division, soliciting, reviewing and editing papers for the Division Journal. Other members of this committee are John M. Buckley, Joseph M. Caldwell and Jay V. Hall, Jr.

New Division Committee Members

Committee on Publications: Col. Ellsworth I. Davis replaces Evan W. Vaughan as Chairman, leaving a vacancy.

Committee on Cooperation with Local Sections: Norman R. Moore replaces Samuel D. Stickle as Chairman.

Committee on Session Programs: Loren W. Olmstead, Raymond W. Sauer and W. Mack Angas replace Gilbert M. Dorland, Donald H. McCroskey and Charles M. Wellons.

Committee on Coastal Engineering: James R. Ayers replaces Harry C. Locher

Committee on Navigation and Flood Control Facilities: Richard V. Meland replaces William E. Kindel.

Substantial Increase in Division Membership

Membership in the Waterways and Harbors Division has increased to about 450, more than double the membership of only a few years ago. A coupon is provided in each issue of CIVIL ENGINEERING for Society members to enroll in two divisions.

Note: No. 1956-27 is part of the copyrighted Journal of the Waterways and Harbors Division of the American Society of Civil Engineers, Vol. 82, WW 5, December, 1956.

Maryland Section Meeting

The Waterways and Harbors Committee of the Maryland Section will sponsor the February 1957 meeting of the Section, which will feature proposed improvements to Baltimore Harbor. Mr. John T. Starr will act as chairman of the technical portion of the meeting and the speakers will be Colonel Stephen E. Smith, District Engineer, Baltimore District, Corps of Engineers, who will talk about the history and development of the Port of Baltimore. Mr. Michael A. Kolesser, Chief of the Navigation Reports Section of the Planning and Reports Branch of the Baltimore District, will speak on the proposed improvements, with particular attention to the design of channels, trends in the construction of super-ships and origin of bulk traffic coming to Baltimore. Both Colonel Smith and Mr. Kolessar are members of the Waterways and Harbor Committee of the Maryland Section.

Valuable Program Presented by Division in Pittsburgh

Four sessions were presented by the Waterways and Harbors Division during the Pittsburgh Annual Convention of ASCE, on October 15 and 16. Twenty-four authors were involved in the program presented, with detailed arrangements being made by Col. Charles M. Wellons, representative of the Committee on Session Programs for the Pittsburgh sessions. Col. Wellons also represented the Executive Committee as presiding officer for the Pittsburgh sessions.

Jackson Convention

The Committee on Session Programs of the Waterways and Harbors Division has scheduled two sessions, to be held on February 18, 1957, for the Jackson Convention. In addition, a luncheon, jointly sponsored by the Waterways and Harbor Division and the Hydraulics Division, will be held on the 18th, and will be addressed by Major General John R. Hardin. Capt. Lewis C. Coxe, Chairman, Executive Committee, Waterways and Harbors Division, will preside at both sessions and the luncheon. The program includes papers on telemetering of hydrologic data, application of computers to hydraulic problems, the Mississippi and Missouri Rivers, Mobile harbor, and industrial development. A complete program will be given in the January 1957 issue of Civil Engineering. The program was arranged by Mr. Raymond W. Sauer and Col. Lawrence B. Feagin.

Meeting at Princeton University during October 1957 Convention

A one-day meeting, sponsored by the Waterways and Harbors Division, will be held at Princeton University on Friday, October 18, 1957, during the New York Convention. The meeting will include three papers by the staff of the Port of New York Authority, describing their marine terminal work. Additional papers will be presented by the faculty of Princeton University. A luncheon will be held at the Princeton Inn. The suggestion for this meeting was made by Adm. William M. Angas, Chairman of the Department of Civil Engineering at the University, who has been named to the Division Committee on Session Programs to assist in the preparation of the program.

Future Conventions

Future conventions are scheduled as follows:

Buffalo, N.Y.	— June 1957	Washington, D.C.	— October 1958
New York, N.Y.	— October 1957	Los Angeles, Calif.	— February 1959
Chicago, Illinois	— February 1958	Cleveland, Ohio	— June 1959
Portland, Oregon	— June 1958		

The Division is planning five sessions for the Buffalo Convention. Arrangements are being made by Col. Loren W. Olmstead.

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each paper number; the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and orders [illegible], refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 861 is identified as 861 (SM1) which indicates that the paper is contained in Issue 1 of the Journal of the Soil Mechanics and Foundations Division.

VOLUME 81 (1955)

DECEMBER: 842(SM), 843(SM)[c], 844(SU), 845(SU)[c], 846(SA), 847(SA), 848(SA)[c], 849(ST)[c], 850(ST), 851(ST), 852(ST), 853(ST), 854(CO), 855(CO), 856(CO)[c], 857(SU), 858(ED), 859(BD), 860(BD).

VOLUME 82 (1956)

JANUARY: 861(SM1), 862(SM1), 863(EM1), 864(SM1), 865(SM1), 866(SM1), 867(SM1), 868(HW1), 869(ST1), 870(EM1), 871(HW1), 872(HW1), 873(HW1), 874(HW1), 875(HW1), 876(EM1)[c], 877(HW1)[c], 878(ST1)[c].

FEBRUARY: 879(CP1), 880(HY1), 881(HY1)[c], 882(HY1), 883(HY1), 884(IR1), 885(SA1), 886(CP1), 887(SA1), 888(SA1), 889(SA1), 890(SA1), 891(SA1), 892(SA1), 893(CP1), 894(CP1), 895(PO1), 896(PO1), 897(PO1), 898(PO1), 899(PO1), 900(PO1), 901(PO1), 902(AT1)[c], 903(IR1)[c], 904(PO1)[c], 905(SA1)[c].

MARCH: 906(WW1), 907(WW1), 908(WW1), 909(WW1), 910(WW1), 911(WW1), 912(WW1), 913(WW1)[c], 914(ST2), 915(ST2), 916(ST2), 917(ST2), 918(ST2), 919(ST2), 920(ST2), 921(SU1), 922(SU1), 923(SU1), 924(ST2)[c].

APRIL: 925(WW2), 926(WW2), 927(WW2), 928(SA2), 929(SA2), 930(SA2), 931(SA2), 932(SA2)[c], 933(SM2), 934(SM2), 935(WW2), 936(WW2), 937(WW2), 938(WW2), 939(WW2), 940(SM2), 941(SM2), 942(SM2)[c], 943(EM2), 944(EM2), 945(EM2), 946(EM2)[c], 947(PO2), 948(PO2), 949(PO2), 950(PO2), 951(PO2), 952(PO2)[c], 953(HY2), 954(HY2), 955(HY2)[c], 956(HY2), 957(HY2), 958(SA2), 959(PO2), 960(PO2).

MAY: 961(IR2), 962(IR2), 963(CP2), 964(CP2), 965(WW3), 966(WW3), 967(WW3), 968(WW3), 969(WW3), 970(ST3), 971(ST3), 972(ST3)[c], 973(ST3), 974(ST3), 975(WW3), 976(WW3), 977(IR2), 978(AT2), 979(AT2), 980(AT2), 981(IR2), 982(IR2)[c], 983(HW2), 984(HW2), 985(HW2)[c], 986(ST3), 987(AT2), 988(CP2), 989(AT2).

JUNE: 990(PO3), 991(PO3), 992(PO3), 993(PO3), 994(PO3), 995(PO3), 996(PO3), 997(PO3), 998(SA3), 999(SA3), 1000(SA3), 1001(SA3), 1002(SA3), 1003(SA3)[c], 1004(HY3), 1005(HY3), 1006(HY3), 1007(HY3), 1008(HY3), 1009(HY3), 1010(HY3)[c], 1011(PO3)[c], 1012(SA3), 1013(SA3), 1014(SA3), 1015(HY3), 1016(SA3), 1017(PO3), 1018(PO3).

JULY: 1019(ST4), 1020(ST4), 1021(ST4), 1022(ST4), 1023(ST4), 1024(ST4)[c], 1025(SM3), 1026(SM3), 1027(SM3), 1028(SM3)[c], 1029(EM3), 1030(EM3), 1031(EM3), 1032(EM3), 1033(EM3)[c].

AUGUST: 1034(HY4), 1035(HY4), 1036(HY4), 1037(HY4), 1038(HY4), 1039(HY4), 1040(HY4), 1041(HY4)[c], 1042(PO4), 1043(PO4), 1044(PO4), 1045(PO4), 1046(PO4)[c], 1047(SA4), 1048(SA4)[c], 1049(SA4), 1050(SA4), 1051(SA4), 1052(HY4), 1053(SA4).

SEPTEMBER: 1054(ST5), 1055(ST5), 1056(ST5), 1057(ST5), 1058(ST5), 1059(WW4), 1060(WW4), 1061(WW4), 1062(WW4), 1063(WW4), 1064(SU2), 1065(SU2), 1066(SU2)[c], 1067(ST5)[c], 1068(WW4)[c], 1069(WW4).

OCTOBER: 1070(EM4), 1071(EM4), 1072(EM4), 1073(EM4), 1074(HW3), 1075(HW3), 1076(HW3), 1077(HY5), 1078(SA5), 1079(SM4), 1080(SM4), 1081(SM4), 1082(HY5), 1083(SA5), 1084(SA5), 1085(SA5), 1086(PO5), 1087(SA5), 1088(SA5), 1089(SA5), 1090(HW3), 1091(EM4)[c], 1092(HY5)[c], 1093(HW3)[c], 1094(PO5)[c], 1095(SM4)[c].

NOVEMBER: 1096(ST6), 1097(ST6), 1098(ST6), 1099(ST6), 1100(ST6), 1101(ST6), 1102(IR3), 1103(IR3), 1104(IR3), 1105(IR3), 1106(ST6), 1107(ST6), 1108(ST6), 1109(AT3), 1110(AT3)[c], 1111(IR3)[c], 1112(ST6)[c].

DECEMBER: 1113(HY6), 1114(HY6), 1115(SA6), 1116(SA6), 1117(SU3), 1118(SU3), 1119(WW5), 1120(WW5), 1121(WW5), 1122(WW5), 1123(WW5), 1124(WW5)[c], 1125(BD1)[c], 1126(SA6), 1127(SA6), 1128(WW5), 1129(SA6)[c], 1130(PO6)[c], 1131(HY6)[c], 1132(PO6), 1133(PO6), 1134(PO6), 1135(BD1).

c. Discussion of several papers, grouped by Divisions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1957

PRESIDENT

MASON GRAVES LOCKWOOD

VICE-PRESIDENTS

Term expires October, 1957:
FRANK A. MARSTON
GLENN W. HOLCOMB

Term expires October, 1958:
FRANCIS S. FRIEL
NORMAN R. MOORE

DIRECTORS

Term expires October, 1957:
JEWELL M. GARRELTS
FREDERICK H. PAULSON
GEORGE S. RICHARDSON
DON M. CORBETT
GRAHAM P. WILLOUGHBY
LAWRENCE A. ELSENER

Term expires October, 1958:
JOHN P. RILEY
CAREY H. BROWN
MASON C. PRICHARD
ROBERT H. SHERLOCK
R. ROBINSON ROWE
LOUIS E. RYDELL
CLARENCE L. ECKEL

Term expires October, 1959:
CLINTON D. HANOVER, Jr.
E. LELAND DURKEE
HOWARD F. PECKWORTH
FINLEY B. LAVERTY
WILLIAM J. HEDLEY
RANDLE B. ALEXANDER

PAST-PRESIDENTS

Members of the Board

WILLIAM R. GLIDDEN
ENOCH R. NEEDLES

EXECUTIVE SECRETARY
WILLIAM H. WISELY

TREASURER
CHARLES E. TROUT

ASSISTANT SECRETARY
E. L. CHANDLER

ASSISTANT TREASURER
CARLTON S. PROCTOR

PROCEEDINGS OF THE SOCIETY

HAROLD T. LARSEN
Manager of Technical Publications

PAUL A. PARISI
Editor of Technical Publications

COMMITTEE ON PUBLICATIONS

JEWELL M. GARRELTS, *Chairman*

HOWARD F. PECKWORTH, *Vice-Chairman*

E. LELAND DURKEE
MASON C. PRICHARD
R. ROBINSON ROWE
LOUIS E. RYDELL